JOURNAL *of the*
BOSTON SOCIETY
OF
CIVIL ENGINEERS

107 YEARS
1848-1955

OCTOBER - 1955

VOLUME 42 NUMBER 4

BOSTON SOCIETY OF CIVIL ENGINEERS

OFFICERS, 1955-1956

PRESIDENT

EDWIN B. COBB

VICE PRESIDENTS

DONALD W. TAYLOR (Term expires March, 1956)	JOHN G. W. THOMAS (Term expires March, 1957)

SECRETARY	TREASURER
ROBERT W. MOIR	CHARLES O. BAIRD, JR.

DIRECTORS

CHARLES H. NORRIS JOHN S. BETHEL, JR. (Term expires March, 1956)	THOMAS A. BERRIGAN ERNEST L. SPENCER (Term expires March, 1957)

PAST PRESIDENTS

EMIL A. GRAMSTORFF
CHESTER J. GINDER
MILES N. CLAIR

SANITARY SECTION

ARIEL A. THOMAS, Chairman
JOHN F. FLAHERTY, Clerk

STRUCTURAL SECTION

CASIMIR J. KRAY, Chairman
JOHN M. BIGGS, Clerk

HYDRAULICS SECTION

RALPH S. ARCHIBALD, Chairman
CLYDE W. HUBBARD, Clerk

SURVEYING AND MAPPING SECTION

ALEXANDER J. BONE, Chairman
ERNEST A. HERZOG, Clerk

Editor—CHARLES E. KNOX

88 Tremont Street, Room 715, Boston 8, Massachusetts

JOURNAL OF THE
BOSTON SOCIETY
OF
CIVIL ENGINEERS

Volume 42 OCTOBER, 1955 Number 4

PAPERS AND DISCUSSIONS

	PAGE
Development of Prestressed Concrete in Bridge Construction. *J. C. Rundlett*	293
Improvements to New Bedford's Sewerage Facilities, Including Intercepting Sewers and Pumping Stations. *Frank L. Heaney*	323
Discussion. *Harold Allen Thomas*	341
Discussion. *Robert T. Jones*	343
Sanitary Engineering in Alaska. *Clarence I. Sterling* (Water)	345
Graphical Solution for Torsional Problems Hydrodynamical Analogy. *Alexander Wargon*	364

OF GENERAL INTEREST

News of Members	374

Journal of Boston Society of Civil Engineers is indexed regularly by Engineering Index, Inc.

Copyright, 1955, by the Boston Society of Civil Engineers
Entered as second-class matter, January 15, 1914, at the Post Office
at Boston, Mass., under Act of August 24, 1912

Published four times a year, January, April, July and October, by the Society
715 Tremont Temple, Boston, Massachusetts

Subscription Price $5.00 a Year (4 Copies)
$1.25 a Copy

Digitized by the Internet Archive
in 2023

https://archive.org/details/civil-engineering-practice_1955-10_42_4_0

JOURNAL OF THE

BOSTON SOCIETY OF CIVIL ENGINEERS

Volume 42 OCTOBER, 1955 Number 4

DEVELOPMENT OF PRESTRESSED CONCRETE IN BRIDGE CONSTRUCTION

BY J. C. RUNDLETT,* Member

[Presented at a meeting of the Structural Section of the Boston Society of Civil Engineers, held on May 11, 1955.]

THE acceptance of a new design is generally not universal unless there is the urgent need of it for practical or economic reasons. Hence, development in one country will lag behind that in another and development within a country such as ours will not be uniform for the same reasons.

Thus, in prestressing, Europe and South America have made the greatest strides in this field spurred on by the scarcity and excessive cost of structural steel and the abundance and cheapness of labor. In this country the initial development was in the northeast during a critical material shortage, but the greater development is now in the south and west where the use of structural concrete is more universal and where 36″ wide flange steel beams are not so readily available. This is largely true in the California area where these members are not rolled and consequently must be shipped in from the east when needed. The natural tendency in that area, because of this and the climatic conditions, is to the extensive use of concrete and thence to the greater use of prestressed concrete.

This is not to say that concrete is not suitable to the widely changing climate of the northeast—it is—but there are other climates where exposed concrete can be expected to have a longer life. For instance, our New England climate is not suitable for the use of concrete piles in sea water, but Florida uses them successfully in all their long trestle work; therefore, the development and the use of prestressed concrete piles in that area is as natural as the greater development of prestressed bridges.

---

*Bridge Engineer, Mass. Dept. Public Works, Boston.

It cannot be expected that prestressed concrete will be universally to bridge construction what it is hoped that the Salk vaccine will be to infantile paralysis.

Public agencies, such as the Department of Public Works in Massachusetts, have done much to lead in the development and testing of prestressed concrete, and their experience has been of a great benefit to the entire building industry. Public agencies can more easily experiment in such work because they are both the engineer and the owner, an enviable position. The public engineer has only his own department head to convince that such work is practical and that it has a future. He does not have to exaggerate to investors that there will be a 10 per cent return on the first installation. He has only his conscience to convince that this is a safe material, that the anticipated cost is not too great, and that continued use will reduce the cost to a competitive level.

The use of prestressing in bridge work is by no means limited to new construction or, for that matter, to concrete. Inadequate structural steel floor beams can be cheaply strengthened by prestressing with cables or rods on each side of the lower portion of the beam, with the force applied against a plate welded to the end of the floor beam.

Inadequate concrete T-beams can be similarly strengthened to raise their capacity to the required loading (Figure 1). Such prestressing against an end plate and at a predetermined distance near the bottom of the beam can, if necessary, reduce the dead load stress in the reinforcing steel to zero and greatly reduce the compression in the top flange of the T-section, thus making available that stress range for live load.

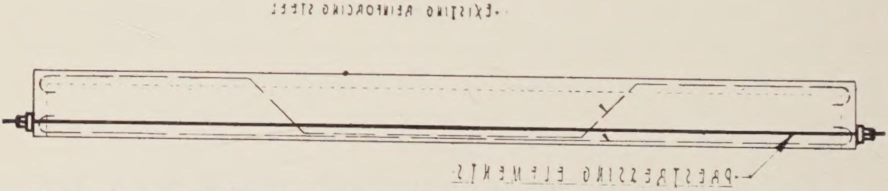


Fig. 1.—Method of Increasing Capacity of Concrete Tee Beam by Prestressing.

In the prestressing of the concrete T-beam, tension may be developed in the upper ends of the beams, but you will recall that most older designs required about half of the bottom steel in a concrete beam be bent up and carried to the ends of the beam. This bent-up steel is in the convenient position then of being able to take the induced tension.

Prestressing is a fascinating phenomenon and its use seems unlimited. Witness the construction of Europe's longest suspension bridge to be built over the Rhine River in Germany. This unique structure, which was written up in the April 7, 1955 issue of the Engineering News Record, has a length of 1,860 feet, and the floor system, which is made up of two exterior stiffening girders and interior stringers, was made composite with the concrete slab. Initially, the stringers were tensioned in such a manner that compression was induced in the stiffening girders. The continuous concrete deck was poured, the tension in the stringers was slowly released, and finally, after about three months, when the shrinkage and plastic flow of the concrete had practically terminated, the stringers were subjected to compression, putting tension in the stiffening girders. In the final state, only bending moments put stress into the girders.

In addition, the concrete slab was prestressed with rods, both longitudinally and transversely, thus preventing any cracks in this long and wide jointless slab.

Development in this country has been, to a large extent, independent of European practice, one notable exception being the Walnut Lane Bridge in Philadelphia. The Magnel method stressing was used on this bridge where the wires are stretched two at a time and fastened with metal wedges.

Following the natural tendency of American engineers the casual comment was that there must be cheaper ways of post-tensioning stressing without having to stress the wires individually. In Europe, further developments had been made along this same line.

In England, Lee-McCall developed a cold worked high-tensioned rod with an ultimate strength of 145,000 p.s.i. These rods, which are available in sizes up to 1⅛″ in diameter, have tapered threaded ends and nut fittings which will develop the practical strength of the bar itself. The Stressteel Corp. of Wilkes Barre, Pennsylvania, has since taken over the manufacture of these rods in this country and are now in the process of developing a swedge fitting to replace the nut.

Freyssinet also developed a relatively inexpensive reinforced cone which allowed the stressing and anchoring of twelve 0.196″ wires in one operation.

In America, Reobling developed a fitting for their bridge strands which made possible the stressing of the multitudinous wires in a 1-9/16″ diameter cable at one time. These cables have an ultimate of 200,000 p.s.i.

The Prestressed Concrete Corp. developed a method of stressing six 0.25 inch wires by threading the wires through holes in a round washer with the head of the wires cold pressed into a button head which bore on the washer, thus allowing six wires to be stressed in one operation.

Another development along the same line was the cold working of two button heads, one at the end of the wire for the stressing operation, and the second near the end for anchoring.

A more recent development of rods has been made by the Tenchen Corp. of Boston. These rods, having an ultimate of 180,000 p.s.i., are heat treated with upset threaded ends which are rated at least 17.5 per cent stronger than the bar itself. The upsets are provided with standard threads and high tensile heat treated nuts. Because of limits imposed by the method of manufacture, the bars are available only in shorter lengths and have to be coupled for long spans.

Rather than continuing with long span structures, as might have been expected, the engineers in this country almost immediately concentrated on the design and production of smaller spans, probably with the thought uppermost in their minds that this is a production country, that to be sold it must be competitive, to be competitive it must be mass produced, and when mass produced it must be transportable.

The first United States Conference on Prestressed Concrete held at M.I.T. in 1951, with its open discussions of extensive European experience and practice and the more limited American developments, stirred the imagination of those in attendance to at least experiment with this new structural phenomenon.

The shortage of structural steel for bridges at about the same time gave the added incentive needed, and so the way was paved for the first new reinforced concrete design since the rigid frame came into prominence about 30 years ago.

Since 1951, with few exceptions, the development has continued in the moderate length transportable-sized beam class, trying out all various type of stressing elements, experimenting with pretensioning and post-tensioning, and with combinations of both, combinations of longitudinal and lateral prestressing, all seeking the simplest, cheapest and most practical method of stressing and the most favorable shape to obtain the lightest weight of beam.

Before proceeding with representative examples of prestressing I would like to speak briefly on design criteria, design details, and testing by the Massachusetts Department of Public Works and the Massachusetts Institute of Technology.

Design Criteria

The only design criteria published to date in this country has been issued by the Bureau of Public Roads. The first specification for design was formulated by that organization in November of 1951. This was followed by a more complete specification in March of 1952 and by the most recent in the latter part of 1954. The earlier publication covered only post-tensioned design, while the latest includes specifications for pretensioned work.

A joint committee of delegates of the American Society of Civil Engineers and the American Concrete Institute is now working on a Manual of Prestressed Concrete which will cover design and construction for both bridges and buildings. This is being written from the composite experience of all those who have been instrumental in its development and who have worked on its design, construction and testing.

The following table shows a comparison of the design requirements between the 1952 and 1954 Criteria.

Thus, it can be seen that the tendency has been to increase the allowable stress in the steel, to allow higher strength concrete, to increase the allowable initial stress in the concrete, to omit the cracking load requirements, and to decrease the ultimate load requirements.

PRESTRESSED CONCRETE DESIGN CRITERIA FOR BRIDGES—AASHO POST-TENSIONED

	1952	1954
Concrete strength	f'c = 4000	Not stipulated
Initial compression stress	0.5 f'c	0.55 f'c'i *
Initial tensile stress	0.04 f'c	0.05 f'c'i *
Design stress after creep and shrinkage	0.4 f'c	0.4 f'c
Tension allowed in extreme bottom fibre	0	0
Initial steel stress	0.6 f'c	0.8 f's
Loss due to creep, shrinkage, etc.	15%	3000 + 11 fcs ** + 0.04 fsi ***
Cracking load	Permissible tension 0.15 f'c = 600 C.L. at least equal to to 1 D.L. + 2 L.L.	Omitted
Ultimate load	2.25 (D.L. + L.L) or 1.0 D.L. + 3.5 L.L. whichever is greater	D.L. + 3 L.L. or 2 (D.L. + L.L.) whichever is greater
Principal Tensile Stresses:		
Design Loads	0.04 f'c	0.03 f'c to be carried by concrete, and excess by stirrup.
Ultimate Loads	0.5 modulus of rupture. If stresses exceed the above, stirrups are to take total shear.	Without stirrups 0.08 f'c. If this stress exceeded, stirrups are to take total shear.
Stirrup Requirements	⅜″ stirrups required regardless of stress, spaced not to exceed ½ depth of beam.	Stirrups recommended, spaced not to exceed ¾ depth of beam.

---

*f'ci ultimate cylinder strength of concrete at time of prestressing (p.s.i.).

**fcs concrete stress at centroid of prestressing steel (p.s.i.).

***fsi initial prestress (p.s.i.).

Undoubtedly, there will be further modifications in the Manual now under preparation, and the Bureau's criteria will then, no doubt, be revised to conform.

Design

One of the important features of prestressed design is the fact that the initial stresses induced at the time of prestressing are never again exceeded except under heavy over-loads.

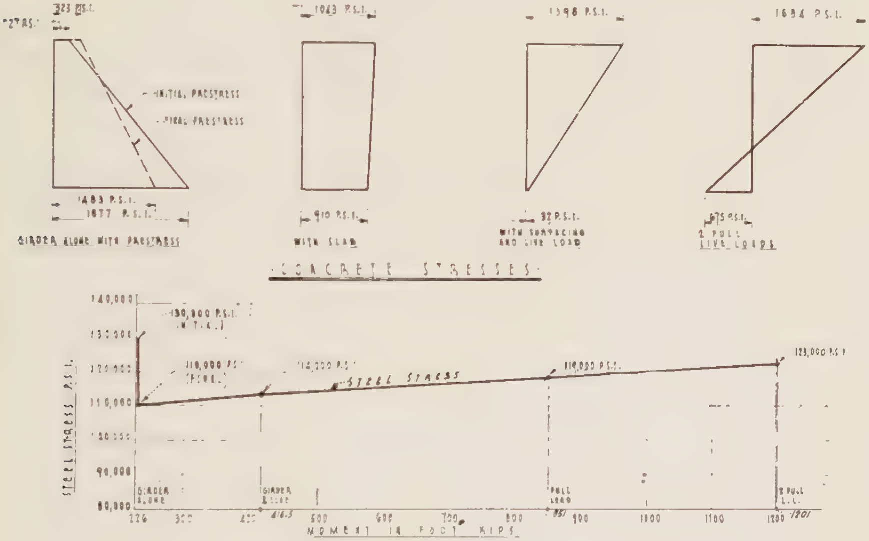


Fig. 2.—Stress Distribution Curves—Hale Street Bridge.

Figure 2 made for the Hale Street Bridge indicates the stress stages in these beams under various conditions of construction and loading.

For the concrete, the initial compression stresses are 1877 p.s.i. in the bottom flange and 227 p.s.i. in the top flange. After allowance for shrinkage, creep, etc., the bottom flange is reduced to 1483 p.s.i., and the top flange stress is increased to 323 p.s.i. With the pouring of the slab the compression in the bottom flange is 910, and in the top 1043 p.s.i.

With the surfacing and design live load the compression in the bottom flange is reduced to 32 p.s.i., while in the top flange it is increased to 1398 p.s.i. With the application of two live loads there is tension in the bottom flange of 675 p.s.i and the compression in the top flange is increased to 1684 p.s.i., which is still about 200 p.s.i. less than the initial compression stress in the bottom flange.

The initial stress in the steel at the time of stressing is 130,000 p.s.i. After loss of shrinkage, creep, etc., this stress is reduced to 110,000 p.s.i. With the poured slab and surfacing added, the stress in the steel is increased to 119,000 p.s.i. under the design live load, and to 123,000 p.s.i. under two live loads.

This is a simple demonstration of a manufactured beam which is prestressed well beyond twice its live load design during its construction. If there are then no visible defects or failures in the beam or in its prestressing elements there will be none under normal live loads or in loads far in excess of normal.

The Department's designs have been very conservative, and I believe in the initial stages of such work that they should be. In the light of our experiences and tests and in view of the new specifications, our costs of this work can be materially decreased.

Diaphragm Details

Diaphragms are required on all prestressed work but, because of the necessity of having uniformity in precast members to make their production economical, the tendency is away from the requirements that they be placed at right angles to the beams except on a square bridge which, incidentally, is now an oddity.

In our early structural designs, diaphragms were believed to be of the utmost importance, that they must always be at right angles to the beams and that they should be made up of heavy beam sections with a depth of not less than 6″ less than the depth of beam. It was felt that these members were the prime distributors of the loads from one beam to the other.

Subsequent tests have minimized the importance of diaphragms and have proved that the slab is an adequate distributor. Thus, the diaphragms become, in effect, more of a construction brace, and the skewing of diaphragms is then acceptable in prestressing.

On our Route 1 work the New England Concrete Pipe Company developed a cylindrical diaphragm stub at the diaphragm location which can be rotated to fit the skew of the bridge, thus obtaining the desired uniformity of details.

TESTING

As a part of our co-operative research program with the Massachusetts Institute of Technology many tests have been run, both at the Institute and at the New England Concrete Pipe Company, on beams selected from our different projects and on small-scale beams.

On the Endicott Street project at Danvers, three beams were tested, two with unbonded steel and one with bonded steel. You will notice from the load deflection diagram of these tests that up to the point of cracking, these three beams behaved in an almost identical manner but that beyond this point the bonded beam showed less deflection. This behavior convinced us that post-tensioned work should always be grouted in bridges because of the ability of that type of beam to withstand better the effects of overloads, and further, because of the added protection of the steel against corrosion.

On the Newbury-Newburyport project, one beam was selected from each of the four bridges constructed. Loading to simulate live load and the dead load of the surfacing was applied at the third points by means of two hydraulic jacks (Figure 3). Loads were applied in stages in each test to a point beyond the cracking load, the loads were then released to zero and then re-applied till failure.

Cracks as they appeared on the beams are indicated on the curves of Figure 4 by numbers under the load at which they occurred. You will note in all cases that they are far above the qualifying condition of two times the live load. The appearance of these cracks was noted by an observer and checked with a sonoscope by placing the poles diagonally on opposite sides of the bottom flange near the point where the crack was expected to appear.

Strain gauges were not used in these latter tests except on the Lee-McCall bars. A controversy had developed because of the brittleness of this type of steel, of the effect of notch action in the threads, and of the possibility of greatly increased stress in the bars by eccentric loadings due to inaccuracies in the setting of the bearing plates. Also, while it was known under normal loads that the stress in the bars and fittings would never exceed the initial stresses, the effectiveness of bond by the grouting of a smooth rod was not known.

Four SR4 strain gauges were attached to the four bottom bars near the end plate and as near the nut as possible to measure the changes in stress at this point as the loads were applied. These gauges indicated that the grouting did give sufficient bond to the extent that there was no change in stress at the ends until the loads had reached the equivalent of four live loads. At this point the bond in one bar became ineffective, the stress increased rapidly, and the member failed through the threaded portion just inside the nut. Unfortunately, the test was then stopped. We regret now that it was not run to destruction.

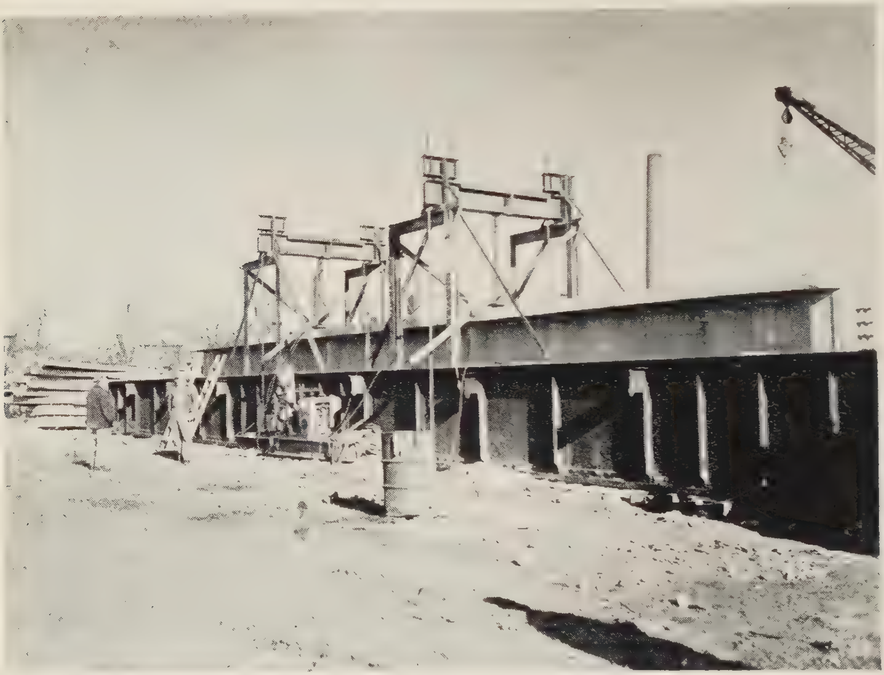


Fig. 3.—Testing Prestressed Beam.

Tests run at M.I.T. on 0.6″ diameter strands further indicated that bond achieved either by grouting or by tensioning the strands prior to casting and supplying end anchorage for the strands after release increased the strand efficiency and ultimate moment by 20 per cent.

A further test was run on a composite steel beam and a slab, with the same span and design conditions as the Freyssinet prestressed beam to compare the action of these two designs. Both load deflection curves have been superimposed on Figure 5 showing an intersecting comparison, and indicating that the prestressed design was superior in withstanding extreme overloads.

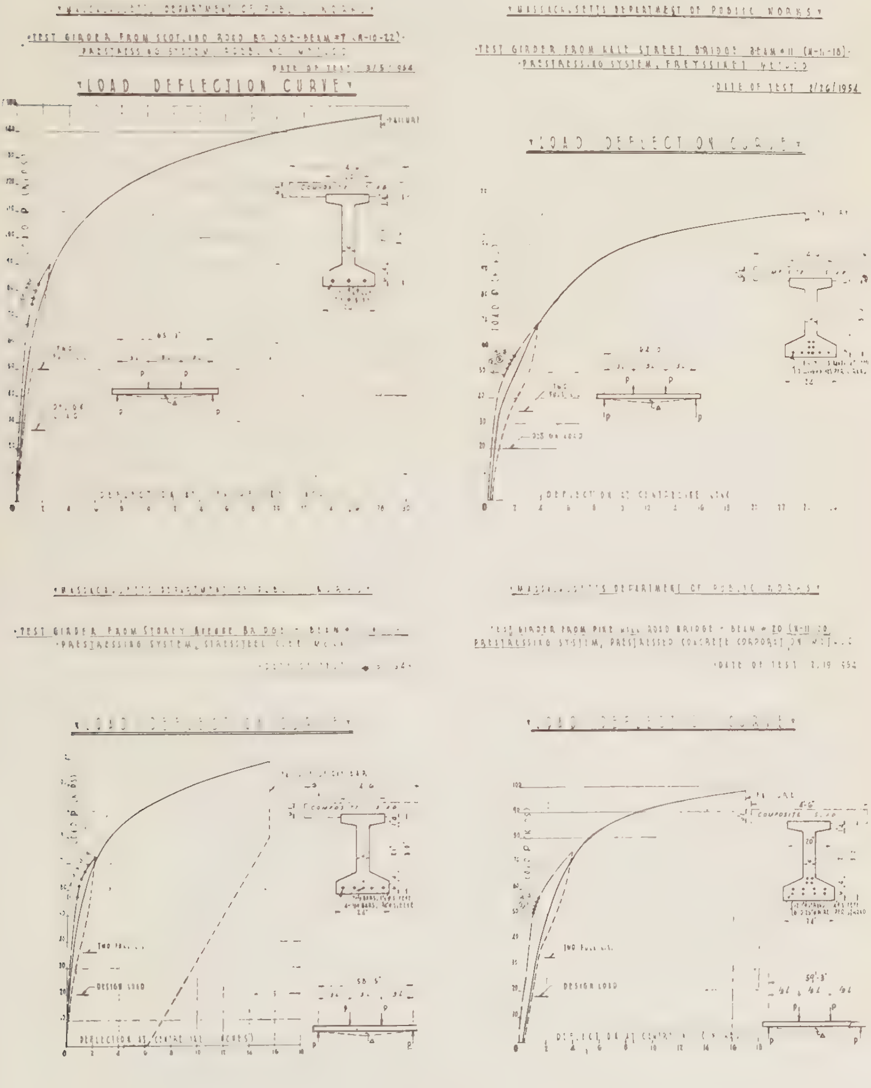


FIG. 4.—LOAD DEFLECTION CURVES.

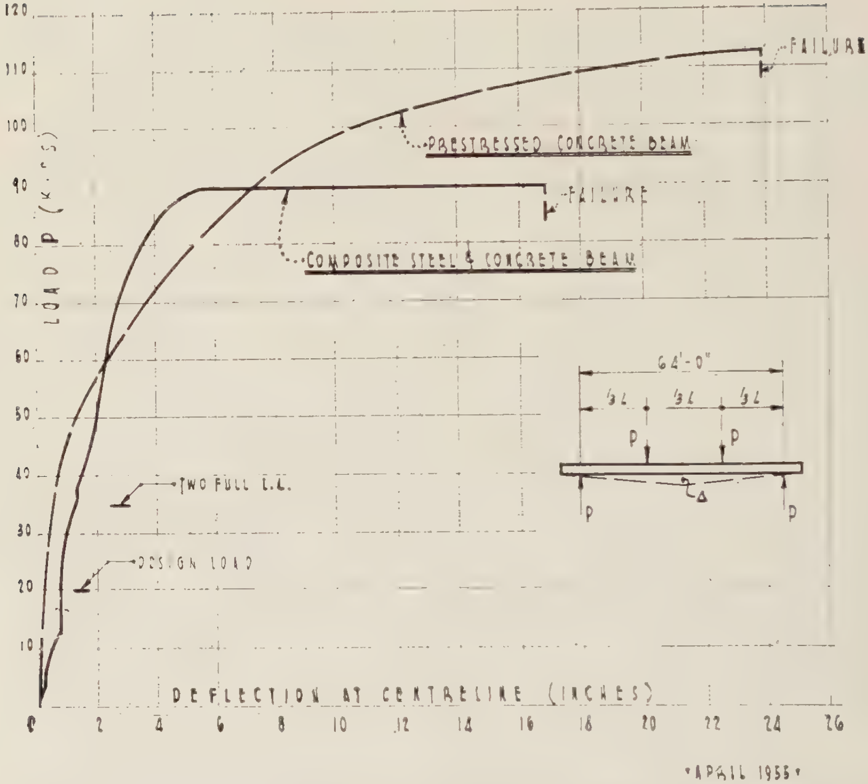


FIG. 5.—COMPARISON OF LOAD TESTS—PRESTRESSED CONCRETE BEAM—COMPOSITE STEEL & CONCRETE BEAM.

The most impressive results of these tests were the flexibility of a prestressed concrete beam, commonly thought to be a rather rigid member. The Scotland Road Bridge, with a span of 65 feet and designed for an H20 S16 loading, had a total deflection of 28″ at failure. The Freyssinet beam, with a span of 64 feet and designed for H20 loading, had a total deflection of 24″, while the Lee-McCall beam, having a span of 58′, and the Prestressed Concrete Company's beam, having a span of 59′, and both designed for H20 loading, had a deflection at failure of about 16″.

The next most impressive result was the almost complete recovery of the beam after the application of about four live loads and the complete closure beyond recognition of any cracks.

These tests, I believe, do not particularly indicate any advantage of one system of prestressing over another, but they do prove beyond any question of doubt that prestressing is an excellent bridge material, that it is structurally sound, and that no one need have any qualms about using it.

Examples of Prestressed Bridges

Turkey Creek Bridge—Tennessee

I believe that the first prestressed bridge in the United States that was open to travel was the Turkey Creek Bridge in Tennessee, designed by Bryan & Dozier and built in 1950. This bridge was of precast block construction similar to the wooden block models used to illustrate prestressing. In this type of design, generally three types of blocks are required; the end or bearing blocks; the intermediate blocks, which retain the cables in position near the bottom of the beam until they are bent up; and the standard plain blocks.

John R Bridge—Michigan

Another similar design to the Turkey Creek Bridge is the John R Bridge designed by Johnson & Anderson and built over the Red River Drain in Michigan in 1951. On this bridge the blocks were constructed with lips and the space between blocks filled with grout. All bridges of this type are stressed with cables after the grout has sufficiently set. Spans have been constructed up to about 32′ and a continuous bridge has been built by using reinforcing steel in the poured slab over the supports.

This type of construction has been very economical and has many advantages, but has not been used extensively. There is still a reluctance to accept beams that are not poured monolithically and, for my part, I feel that they do not have a place in our climate.

Block construction is the reason for the omission of a cracking load requirement in the most recent specification because, in this type, computations for cracking are largely meaningless.

Wyoming County—New York

A later development of multiple units in a beam has been made by Mr. Felix Ramsey, Superintendent of Highways in Wyoming County, New York, in an effort to produce beams using his own limited forces and equipment.

For ease of handling, each beam is precast in three sections with the size of the unit limited by the capacity of his cranes. Each individual section is reinforced sufficiently to allow handling and transporting.

The longest span which he has developed to date is 52 feet, and was cast in about 17′ lengths with two end units and one center unit. The section is T in shape, 3′ deep and has a flange width of 30″.

Each section is poured against the other in the shop to insure an accurate fit in the field. The precasting is done indoors in the winter season and stored there until time for erection. The units are then taken to the site, placed in position on the abutments and on two temporary bents, the wires are threaded through pipe sleeves, stressed, anchored and grouted. The Freyssinet system is being used.

Arroyo Seco Bridge—California

The Arroyo Seco Pedestrian Bridge was built in California in 1950. This structure has a span of 110′, a width of 8′ and is the west's first prestressed bridge. The beams are 5′-8″ deep, with 10″ webs, were cast on the adjacent roadway and moved into place with three cranes. The stressing was done by the Prestressed button head method.

Walnut Lane Bridge—Philadelphia

The first bridge placed under construction in the United States was the Walnut Lane in Philadelphia, which was completed in 1950. It contains three simple girder spans, a center span of 155′, and two end spans of 74′ each. The girders are I-shape, 79″ deep with 52″ flanges. In the center span the flanges are butted, but in the end span the beams are placed 8′-8″ on centers and the slab poured in place. As noted before, this bridge was tensioned by the Magnel method, two wires at a time. The beams were cast on falsework at the bridge site and were moved horizontally into position on the foundation.

Endicott Street Bridge—Danvers

In the fall of 1951, Massachusetts started construction on the first prestressed bridge to be built in northeastern United States. This was the Endicott Street Bridge in Danvers over the Boston and

Maine Railroad. This bridge has a span of 27′-4½″. The beams are of an inverted T-shape with the bottom flanges butted and the area between and over the webs filled with concrete, making in effect a composite slab. The beams are 14″ deep with a 12″ flange and were stressed with three 0.6″ galvanized wire strands, two straight and one parabolic, and all unbonded. The 47 beams were cast in the Providence shop of the New England Concrete Pipe Company, about 65 miles from the bridge site. They were transported by truck over the road and erected all in one day.

The particular advantage of this type of bridge is that no falsework or forms are required for the pouring of the deck slab except as may be necessary for the copings and curbs.

While this type of beam has not been extensively used on bridge work it was used with pretensioned strands on Pier 57 in New York very successfully.

Pretensioned Beams—Pennsylvania

In 1951 the Concrete Products Company of Pottstown, Pennsylvania, having previously manufactured precast inverted channel-shaped units, conventionally reinforced, became interested in prestressed concrete. Their natural interest was in production and they became convinced at the start that only by pretensioning could prestressed work be mass-produced and made competitive.

Pretensioning requires the setting up of a long bed with end blocks, against which the multiple wires or strands which extend the full length of the bed can be stressed. The concrete is then poured around the stressed elements into as many beams as can be accommodated on the bed.

With the time element still in mind, the concrete is cured by the vacuum process and when it has reached a predetermined strength the jacks are released and the wires cut, thereby transferring the induced stress in the strands to the concrete. The net result is a prestressed beam produced with the minimum of fittings, fixtures and man hours.

The beams which they now produce, Figure 6, are standardized with a 3′ width and are butted when in place on the foundation. In order to save concrete, and consequently weight, the beams are hollow.

For spans 18′ to 36′ in length, beams 17″ to 21″ in depth are used, and for spans from 38′ to 50′, 33″ depth beams are used.

In the initial development of pretensioning ¼″ strands were called for, but the tendency now is to increase the size, thereby reducing the number of units required. The size has been increased to ⅜″ with an occasional 7/16″, and the limit, of course, will be that size which can adequately and safely develop the bond of these high stress units.

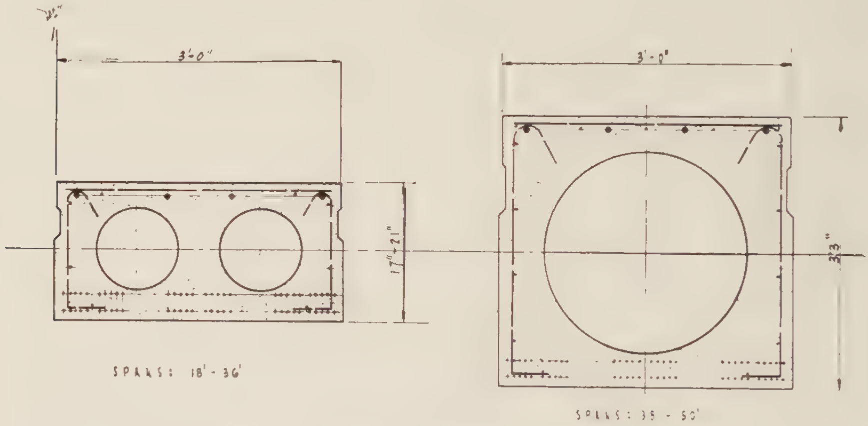


Fig. 6.—Typical Pretensioned Beams.

Beams 36″ deep are now being produced, and plans are being made for beams 42″ deep and 70′ long.

Florida—Tampa Bay

No paper on prestressing would be complete without a reference to the original Tampa Bay Bridge in Florida which was the first and best example of mass-produced prestressed concrete units for bridge work in this country.

Florida is the ideal location for concrete pile trestles and their lengths are measured in miles rather than in feet. The prestressed work on this particular bridge is part of the 15-mile long causeway linking St. Petersburg and the West Coast of Florida. The structure has a roadway width of 28′ and two 3′ sidewalks.

Over three miles of this bridge is of prestressed construction and it is divided into four sections, one of 5800′, one of 10,900′ and two of 325′ each, with spans of 48′. 2,178 beams were required.

Six casting beds, each large enough for the manufacture of 12 beams, were constructed on a spit of made ground projecting into Tampa Bay. These beds were arranged for a continuous operation of casting and stressing. The beams were cast, the forms removed in one or two days and transferred to another bed, and the beams stressed on the seventh day if the cylinder breaks indicated 4,000-pound concrete. The beams were stressed with Lee-McCall bars, then loaded onto barges, towed to the site and transferred to the bents with a floating crane.

Newbury-Newburyport Bridges—Massachusetts

Massachusetts has to date constructed 7 Prestressed Bridges including the one on Endicott Street. The other six made use of an I section with this type of beam substituted for a steel I beam of about the same depth and spacing and calculated for composite action under live load. All of the beams were manufactured by the New England Concrete Pipe Company.

Of particular interest are the 4 bridges built on the Newburyport Turnpike in Newbury and Newburyport, each making use of a different type of stressing element.

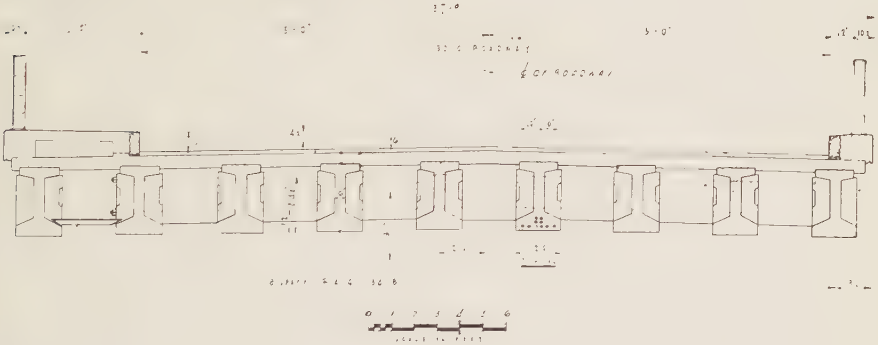


Fig. 7.—Cross Section—Hale Street Bridge.

All the beams are of I shape, Figure 7, having a bottom flange width of 24″, a top flange width of 20″ and a 6″ web. Three have a depth of 3 feet and were designed for H20 Loading while the fourth, the Scotland Road Bridge, had a depth of 3′-8″ and was designed for an H20 S16 loading. The general spacing of beams is about 4′-6″.

The Scotland Road Bridge carrying Route 1 has a span of 65′

and contains 24 beams stressed with three 1-$\frac{9}{16}$″ strands furnished by the American Steel & Wire Company. The Hale Street Bridge, having 2 spans at 64′ contains 18 beams stressed by the Freyssinet system each with 10 prestressing elements of twelve 0.196″ diameter wires. The Storey Avenue Bridge having 2 spans at 58′ contains 36 beams stressed by the Lee-McCall system using four 1⅛″ straight bars and two ⅞″ draped bars. The Pine Hill Road Bridge having 2 spans at 59′ contains 20 beams stressed by the Prestressed Concrete method using twelve 6-wire units of .25″ wires.

All of the beams were grouted. The original specifications called for a different type of enclosure tube for the wires and strands but the contractor used a flexible metal hose for all beams. This allowed the coiling of all the units in the hoses before shipment except for the Lee-McCall bars.

For the 1-$\frac{9}{16}$″ strand, a 2″ diameter hose was used; for the 12 wires of the Hale Street Bridge, a 1¼″; for the 1⅛″ diameter and the ⅞″ diameter bars of the Storey Avenue Bridge, a 1¾″ and 1½″ respectively; and for the 6 wires of the Pine Hill Road Bridge, a 1¼″ hose was used.

The sections cut from the various test beams bear evidence to the success of the grouting operation.

These beams were fabricated out-of-doors during the colder winter months in a prepared bed and shelter. The bed, with width enough for 2 beams to be constructed at one time, was enclosed with two 4′ diameter concrete pipe side walls over which a sectional wooden roof was placed and the ends were enclosed in canvas after the pouring and during the curing.

All the concrete was air-entrained high early with a strength of 5000 p.s.i. and the beams were all steam cured.

One set of steel forms was used in the early stages but after the letting of the Bridgewater contract for 24 more beams, a second set of forms was made.

The diaphragm stubs on these beams as previously described were of a turret type and were spaced equally on all beams, and the end bearing blocks were identical so that the variation in the forms was only the length between the last diaphragm and the end block on all beams and for the depth on one.

The complete manufacture and stressing of a beam was on a 2-day schedule with some interruptions.

A cage containing all the reinforcing steel, stirrups and stressing elements was set up on one side of the bed so that no time was lost in the bed during the fabrication of this unit.

During the pouring of the concrete both internal and external vibrators were used with the net result of perfect beams with no honeycombing. except on one beam made from a new shipment of cement which beam was subsequently destroyed and replaced.

To say that the work progressed without incident would be far from the truth.

The Lee-McCall bars which were shipped in bundles from England arrived unbundled and bent from careless handling. Most of these could be used but a few were replaced. Carelessness in stressing one of the first bars caused the bar to break and the jack to fly, breaking the operator's leg.

Many of the Freyssinet cones arrived in less than perfect shape and had to be replaced and some slipping developed after the stressing, but the amounts were insignificant.

The cold formed button heads on the wires caused some concern because of longitudinal splits. The manufacturer assured us that this was not a weakness and failure of the wire and not the head on subsequent tension tests proved his point.

While the cables for the Roebling Type were prestretched, the coiling of the cables in the tubes had disturbed the lay so that in the first stressing, the strain was not proportional to the stress. This difficulty was overcome by partial stressing and relieving before final stressing.

On our coldest days, when the bed was uncovered, a crack developed in the middle of the top flange apparently due to the rapid change in temperature. This crack in no case extended through the bottom flange and when the beam was stressed the crack closed completely even though camber was induced due to the stressing.

The beams were hauled over the road with one end on a truck and the other end on a dolly and in order to give lateral stiffness, two were shipped at a time. The carrier violated his hauling permit in length and in load and some time was lost in arranging for a new permit.

To cap the climax some one, probably a structural steel man, set fire to a load of hay stored under the Storey Avenue Bridge at 4:45 one morning (Figure 8). Some concrete was spalled on the beams, but load tests indicated that there was no structural damage and that the beams were satisfactory.

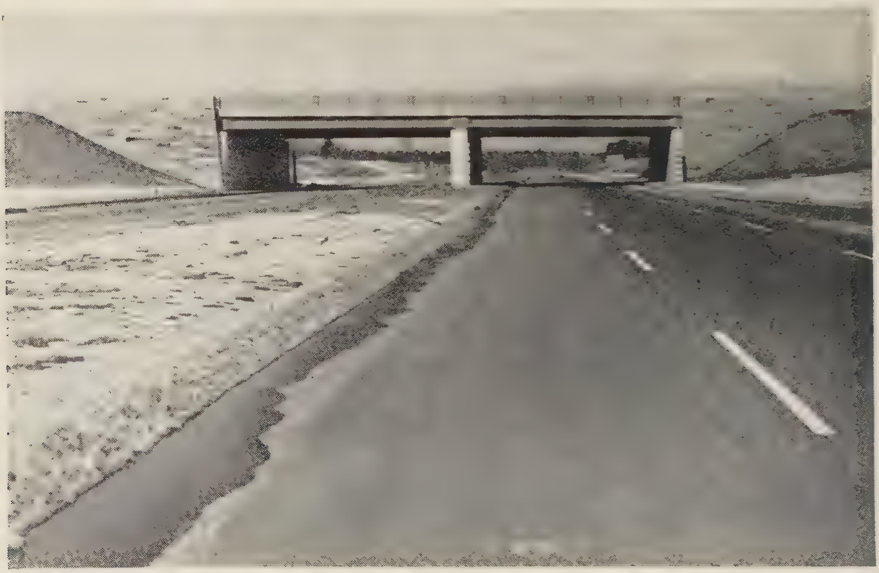


Fig. 8.—Storey Avenue Bridge.

It is in this manner that experience is gained and it is for these reasons that some people feel justified in letting others do the experimenting.

Briefly, I would like to describe a few of the odd shapes and methods that have been used in prestressing.

Hubbard Street Bridge—Connecticut

Connecticut's first prestressed bridge was on Hubbard Street under the Hartford-Glastonbury Expressway.

It has a span of 52′-6″ and is 90 feet wide. There are 20 T-shaped beams in the section, each 2′-6-½″ deep with a flange width of 30″ and a stem width of 15″. The beams are spaced 4′-6″ on centers and are topped with a poured concrete slab which was prestressed transversely.

The superstructure contract was let separately and no specific system was designated, but the steel area, positioning and total tensioning force was given. The Freyssinet System was used.

Atlantic Street Bridge—District of Columbia

The District of Columbia's first prestressed bridge was on Atlantic Street over the Oxon River. This bridge has a span of 74′-9″ with provision on the deck for a 40′ roadway and two 6′ sidewalks.

The nine beams in the structure are 3′-0″ x 3′-3-½″, have a hollow core, and are spaced 6′ on centers and are topped by a conventionally reinforced slab.

The beams were cast near the site and were stressed by the Freyssinet System.

West Mississippi Avenue Bridge—Denver, Colorado

Another interesting development was the bridge built in Denver on W. Mississippi Avenue over the Platte River. This structure is a concrete pile trestle having 4 spans at 50 feet with provisions on the deck for a 40′ roadway and two sidewalks.

The beams have tapered sides with a re-entrant seat for the slab and are spaced 9′-3″ on centers. They are 3′ deep, 3′-6″ wide, contain a 24″ economy hole and were stressed by the Freyssinet System.

The slabs are precast and pretensioned, and finally the structure was laterally post-tensioned.

Two similar examples of fairly long span prestressed construction are the Willow Creek Bridge on the Columbia River Highway in Oregon, Shawan Road Bridge on the Baltimore-Harrisburg Expressway in Maryland.

Willow Creek Bridge—Oregon

The Willow Creek Bridge (Figure 9) has 3 spans at 98′ with provision on the deck for a 30′ roadway. There are 5 modified I-shaped beams in the cross section, spaced 7′-2-¾″ on centers. These beams are 5′-4″ deep, have a 7″ thick web, a bottom flange 2′-2″ wide and a top flange 4′-0″ wide. The slab between the top flanges and diaphragm stubs is cast in place and the entire deck laterally prestressed. The Freyssinet System was used.

Shawan Road Bridge—Maryland

The Maryland Bridge has a span of 100 feet and a roadway width of 30′ and two safety walks. It spans the expressway and was chosen as an experiment in contrast with two other 3-span steel stringer bridges on the same project.

There are 9 beams in the cross section spaced 4′-1-¼″ on centers. The beams are 5′ deep, have an 8″ web, a bottom flange 20″ wide and a top flange 3′-8″ wide. The poured slab between the top flange and the diaphragm stubs in this case was only 5¼″ in width as compared to a width of 3′-3″ on the Willow Creek job.

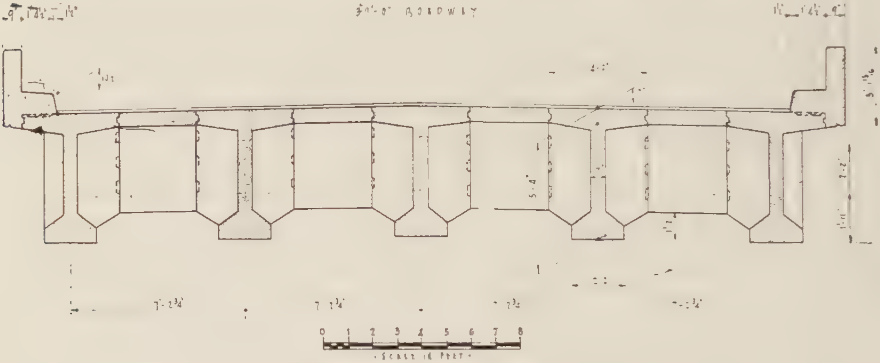


FIG. 9.—CROSS SECTION—WILLOW CREEK BRIDGE.

The Freyssinet System was used on this structure and the bridge was laterally prestressed.

Both of these structures made use of the top flange of the prestressed beams as a part of the structural roadway supporting slab with a bituminous surface for pavement. This type of construction cuts down the amount of concrete work in the field and results in an economical deck. In Massachusetts the deck would have a 3-ply membrane waterproofing course under the surface for protection of the slab.

My own feeling on the matter is that in New England the prestressed units should have the added protection of a poured top slab.

Future Farmers of America Bridge—West Virginia

Another example of pretensioning is the Future Farmers of America Bridge in West Virginia. The designer of this structure departed from the I and the square or rectangular section and chose an inverted U section with the prestressing element located in the vertical legs. This bridge has a span of 65 feet, is designed for an H20 S16 loading and has a depth of 2′-8″.

The precast sections are all 3′ wide and of 2 basic sections; the main roadway sections and the curb sections. These units are butted and bolted together in place and the keys near the tops of the units are grouted. Only the installation of a wearing surface is necessary before the bridge can be opened to traffic.

This is an excellent design but in climate such as ours an application of 3-ply membrane waterproofing would be required under the surfacing. Without it we would be faced with the seepage of water through the butted section and the deterioration of the concrete due to the many cycles of freezing and thawing.

Garden State Parkway Bridge—New Jersey

Twelve bridges on a recently completed section of the Garden State Parkway in New Jersey were of precast and prestressed concrete. The final plans called for the 28 exterior beams to be rectangular and post-tensioned and the 190 interiors to be of a modified I shape and pretensioned, Figure 10.

The spans varied from 39 to 60.4 feet but the beams were all made of two basic cross sections with the prestressing elements varying according to the span. The interior beams are 33″ deep with a 6″ web, 12″ wide top flange and a 19″ bottom flange. Twenty to forty ⅜″ strands were used in these members. The exterior beams are 33″ deep and 16″ wide and are reinforced with units of ten ¼″ wires of the button head type.

These beams were manufactured in three plants about 30 miles from the bridge sites. High early strength cement was used, the concrete was steam-cured, and the beams cast in a two-day cycle, identical with the procedure used for the manufacture of the Massachusetts beams.

The Formigli Corporation, manufacturers of these beams, are now prepared to continue their production for other structures up to 60′ spans within hauling distance of their plants.

Lake Pontchartrain Bridge—Louisiana

The designer's dream of precasting and prestressing an entire span of a bridge is about to come true in Louisiana with the construction of the Lake Pontchartrain Bridge on the Greater New Orleans Expressway. This bridge, which has an over-all length of approximately 23-¾ miles, is a trestle type structure having 2237 spans of 56′ each with provision on the deck for a 28′ roadway.

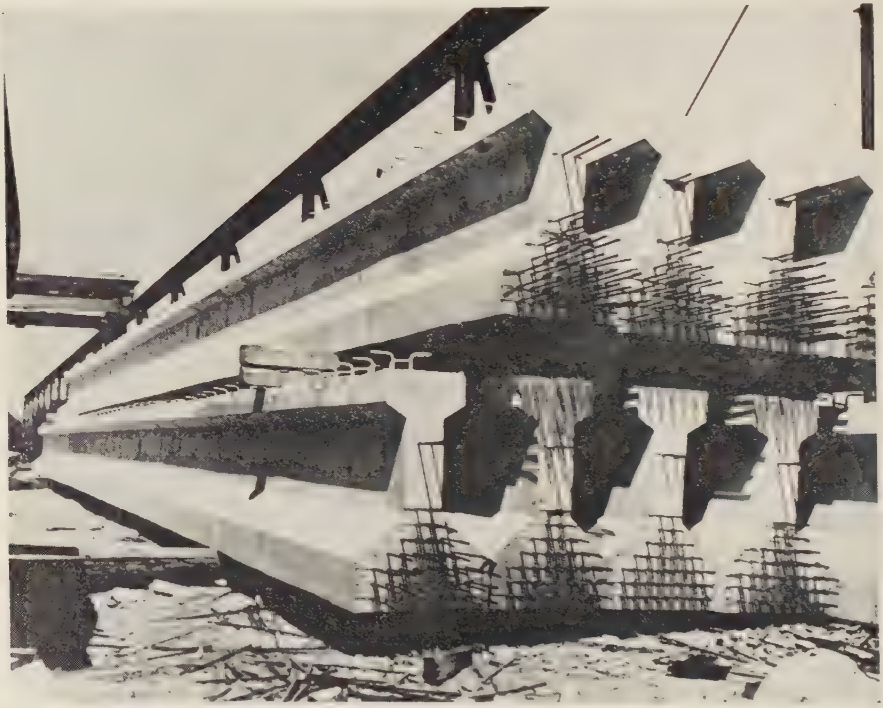


Fig. 10.—Garden State Parkway Beams.

A typical span, Figure 11, is composed of 7 modified I-shaped beams 4′ deep with a 6-¾″ top slab. The 7 beams and the slab are to be precast monolithically as a pretensioned unit. The 5 interior beams are to be stressed with thirty-five 5/16″ diameter wire strands, and the exterior with thirty-three 5/16″ diameter strands. The total weight of one span is about 180 tons and it is to be supported by 54″ prestressed concrete piles.

The piles, which are the cylindrical prestressed type developed by the Raymond Concrete Pile Co., are now being cast by the contractor. The casting bed for the deck has been designed by the Freyssinet Co. and is under construction, but no spans have yet been cast.

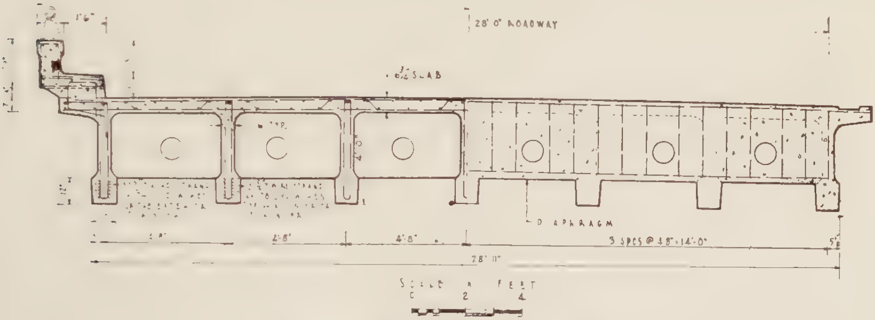


Fig. 11.—Cross Section—Lake Pontchartrain Bridge.

The designers are Palmer & Baker, Inc., of Mobile, Alabama. The contract was awarded to the Louisiana Bridge Co., a combination of Brown & Root and T. L. James, joint venturers.

Hampton Roads—Virginia

On October 8, 1954, bids were received for the construction of a concrete pile trestle type prestressed structure for the Hampton Roads Bridge in Virginia. Alternate bids were requested for 261 post-tensioned beams versus steel stringers for the 80′ spans, and for post-tensioned versus pretensioned for 896 beams in the 50′ spans.

For the 80′ spans, six 1-⅛″ diameter bars were indicated for the 4′-7″ deep beams. For the 50′ spans, four 1-⅛″ diameter bars were shown for post-tensioned and thirty-six ⅜″ strands for the pretensioned alternate. Both of these latter types were 3′-7-½″ deep but a wider beam was required for the pretensioned members to provide ample space for the greater number of prestressing units. 24″ prestressed concrete piles were specified for the bents. The low bid submitted by Merritt, Chapman & Scott was on the prestressed beams and the final selection was for 50′ pretensioned spans for the entire project. Parsons, Brinckerhoff, Hall and MacDonald are the consultants.

Tampa Bay Bridge—Florida

On September 30, 1954, alternate bids were received for the construction of a bridge adjacent to the Gandy Bridge over Tampa Bay in Florida. This proposed structure contains 252-48′ spans and prestressed construction was bid as an alternate to conventional reinforced concrete T Beams. There are also twenty 72′ spans with prestressed construction bid as an alternate with steel I beams.

For the 1505 concrete piles, prestressed or a standard reinforced type are the contractor's choice. An alternate type of prestressing was also indicated. For the 72′ spans, six 1-⅛″ bars were indicated for a post-tensioned beam while the alternate was for twenty-four 7/16″ diameter straight cables to be pretensioned and two 1-⅛″ diameter draped rods to be post-tensioned. For the 48′ spans, four 1-⅛″ bars were shown for the post-tensioned beam and twelve 7/16″ strands for the pretensioned in combination with two 1-⅛″ diameter draped rods to be post-tensioned.

The low bidder submitted a price on only the prestressed design and for that design post-tensioning was selected on all beams using Stressteel bars.

Subsequent to this, another trestle 3000 feet long was bid and the type chosen, which was again the contractor's option, was for combined pre- and post-tensioning, using strands for the pretensioned straight members and Freyssinet units for the post-tensioned draped members.

Prestressed Piles

The prestressing of concrete piles using the pretensioned method is rapidly gaining in favor. One of the disadvantages of long conventionally reinforced piles is the extreme care that must be exercised in handling and in driving because of cracking. Prestressing is the panacea for this. With the concrete in compression, cracking is eliminated in handling and any shrinkage cracks which develop after the pouring are permanently closed.

All the concrete pile projects in Florida bid to date have called for prestressed piles as an alternate to conventionally reinforced piles with the choice being the contractor's, and invariably prestressed piles have been used.

The pile sections in that state have been standardized for 12″, 14″, 16″ and 18″ square sections, reinforced with twelve ⅝″ strands, twelve ⅜″ strands, sixteen ⅜″ strands, and sixteen 7/16″ strands, respectively, with maximum lengths for these sections of 75′, 85′, 95′, and 107′ (Figure 12.)

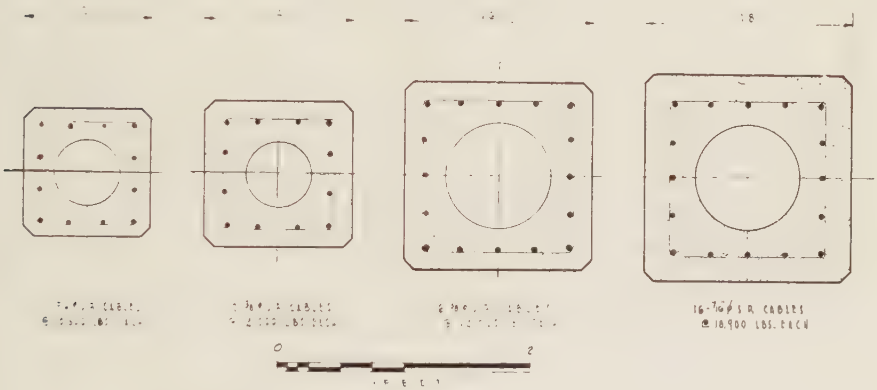


FIG. 12.—TYPICAL PRESTRESSED CONCRETE PILE DETAILS.

Continuous Structures

Continuity is lagging in this country. There is a reluctance to duplicate or even approach the work of Dr. Fritz Leonhardt of Germany whose continuous design allows the stressing of an entire bridge in one jacking operation. Perhaps our reluctance is due to the natural conservatism of our bridge designers who have even been slow in accepting long span steel girders, but more probably the patented features of this design and the license requirements are a retardant to its use.

Two notable examples of Dr. Leonhardt's continuous design are the Railroad Bridge at Heilbrann and the Highway Bridge near Heilbrann over the Neckar Canal.

The Railroad Bridge has 5 continuous spans and is 327′ long. Being a constant depth box girder design, the prestressing elements which are laid in a metal sheath are raised over the 4 supports and at the ends where they are wound around a concrete jacking block. After the cables are laid, the concrete deck is poured up to the jacks on the concrete jacking block. The entire bridge is stressed in one operation and the jacks concreted in.

The most famous of Dr. Leonhardt's designs is the 3-span continuous bridge over the Neckar Canal, Figure 13. It has 2 spans at 62′ and a center span of 312′ with a depth of only 5′-7″ at the center. It is said to be the longest concrete girder span in the world. One of the continuous bridges built in this country is the grade separation structure at Waugh and Memorial Drive in Houston, Texas. This structure, on a sharp curve, has 2 spans at 45′, 1 at 65′ and 1 at 70′ with an over-all length of about 229′. The entire structure is banked. It has a 24″ thick slab containing 58 stressing units in the section with economy holes between each unit.

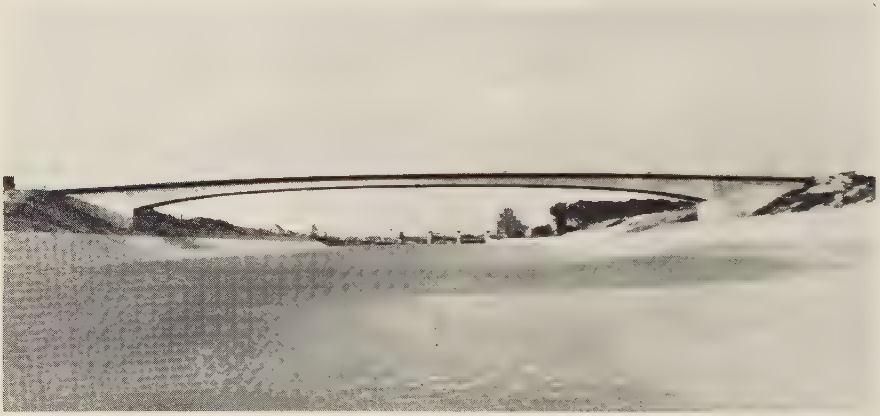


FIG. 13.—NECKAR CANAL BRIDGE.

A similar design was made for the Furnace Brook Parkway Bridge on the Southeast Expressway in Massachusetts, but it was not built because of an entire change in alignment.

Other plans prepared include the Little Falls Bridge in Maryland for the Corps of Engineers. This structure is similar to the Neckar Canal Bridge in that it has a center span of 216′ and cantilevered counter-weighted spans of about 30′. It is a box girder section 7′ deep at the crown, and 12′ deep at the support containing only 2 stressing elements.

A study has also been made for a continuous bridge in Providence over the Providence River having 10 spans with spans varying from 100 to 120′ in length. This, too, is a box girder type.

California

The California Bridge Department has been one of the pioneers in concrete box girder designs. Many of their grade separation bridges (conventionally reinforced) are so designed. These are generally of uniform depth for both simple and continuous spans and are readily adaptable to skews and almost any degree of curvature.

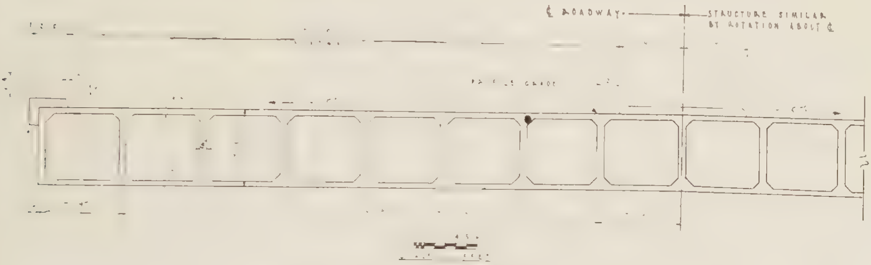


FIG. 14.—HALF CROSS SECTION—BIG DALTON WASH BRIDGE.

The Big Dalton Wash Bridge near El Monte, Figure 14, is of this type but is prestressed. It has a span of 115′ and was designed for H20 and S16 loading. The depth of structure is 7′-3″, the top slab is 6-¼″ thick, the bottom slab is 5-½″ thick and the interior walls are 8″ thick. The stressing elements are in the bottom slab and walls.

Under construction in California is a continuous structure of this same type.

CONCLUSION

It has been an interesting and stimulating experience to have had some part in the development of prestressed concrete, and to have had an opportunity to follow the trends and development of prestressing in this country. In its early stages, it was not comparable in price with steel stringers, but its continued use and the adoption of more economical methods and the discarding of unnecessary features have brought it to the point of being highly competitive and desirable.

Pretensioning is rapidly gaining favor and I believe that this or a combination of pretensioning and post-tensioning will be the logical and economical solution in mass production. I would expect on moderate spans that post-tensioning will, in time, be used only on isolated structures in areas where pretensioning beds have not been set up and on the longer beams which, because of their extreme length and weight, will have to be produced and stressed at the bridge site.

What of the future of prestressing? I believe any good and economical design has a bright future in this country. We are in the midst of a gigantic highway program, the end of which cannot now be seen. Barring a national emergency, the program cannot be halted until the job is done, and that job includes the building of thousands of structures unthought of a few short years ago.

This is a great steel producing country and unless there is a shortage of critical materials, steel will not be replaced in bridge work by prestressing, but prestressing will probably be used far beyond our wildest dreams to help build the highways for an age when our future economy and, perhaps, our eternal existence will depend on our being a highly mobile country.

IMPROVEMENTS TO NEW BEDFORD'S SEWERAGE FACILITIES INCLUDING INTERCEPTING SEWERS AND PUMPING STATIONS

B. Frank L. Heaney,* *Member*

[Presented at a Joint Meeting of the Boston Society of Civil Engineers and Sanitary Section, BSCE, held on December 15, 1954.]

New Bedford, a city with a population of about 110,000, is in southeastern Massachusetts. It is located on a comparatively narrow ridge which forms a peninsula extending into Buzzards Bay. The general location is shown on Figure 1.

Combined sewers to receive both sanitary wastes and storm waters were constructed before 1900. These serve the thickly settled portions and the industrial areas of the city. The combined sewers discharged into tidal waters at the foot of the various streets. Because of this, an offensive condition existed in the waters of the Acushnet River and in Clark's Cove.

Construction of an intercepting sewer system was started in 1912 to eliminate these objectionable conditions. This system was intended to intercept the dry weather flow of the combined sewers and carry these wastes to discharge in Buzzards Bay. An outfall sewer was constructed extending about 3300 feet off the tip of the peninsula forming the city. The depth at the discharge end of the outfall sewer is about 30 feet below mean tide level. Figure 2 shows the route of the intercepting and outfall sewer.

The main intercepting sewer was constructed at as low an elevation as would admit of the handling of sewage by gravity, without resort to pumping. This choice of an intercepting sewer route required that other intercepting sewers and pumping stations be constructed in the nine districts shown on Figure 2. The sewage from these areas lying below the level of the intercepting sewer would thereby be collected and pumped to the intercepting sewer, thence to flow by gravity to discharge in the deep waters of the bay.

In the early 1920's, large textile industries began slowing down operations and many moved away from the city. At the same time, work was halted on the intercepting sewer construction program. Only five of the nine pumping districts had been completed.

---

*Senior Engineer, Fay, Spofford & Thorndike, Engineers, Boston, Massachusetts.

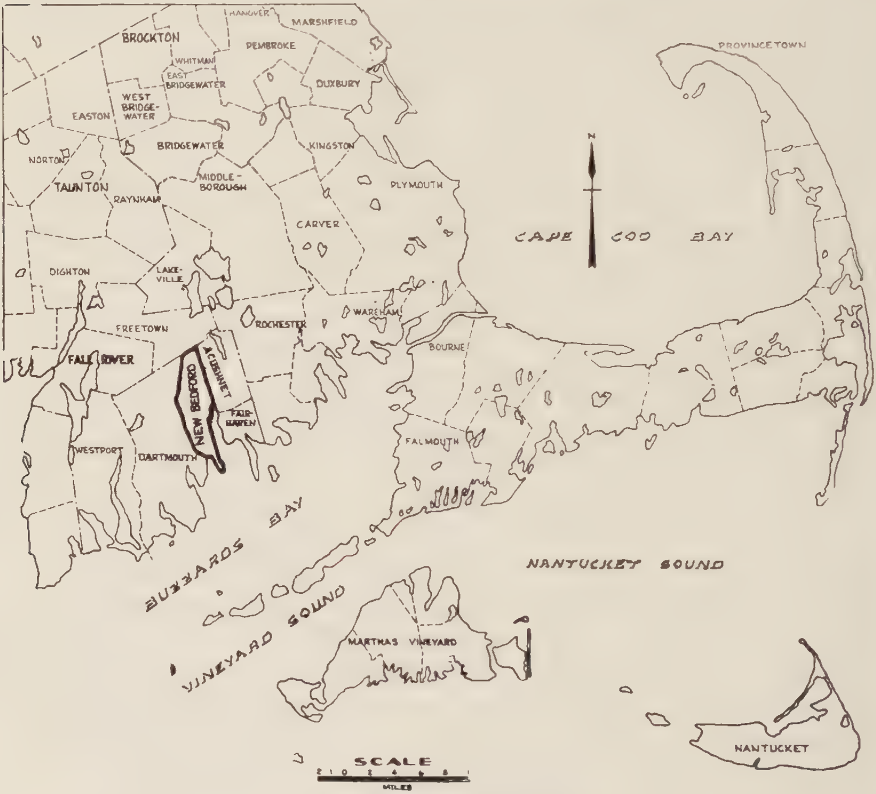


FIG. 1.—GENERAL LOCATION.

The district at Howland Street was served by an underground pumping station. This underground station operated for many years. Maintenance was very difficult and hazardous and it was finally abandoned and filled with sand.

The remaining four districts lying below the level of the intercepting sewer continued to discharge their sewage and industrial wastes into near-by tidal waters of Acushnet River.

Acushnet River is scarcely more than a brook flowing into and forming a tidal estuary. Dispersal of these wastes therefore depended almost entirely upon the flushing action of tidal changes. Floating solids, scum, disagreeable odors, and other evidences of pollution near piers and docks and along New Bedford's fine beaches caused agitation to be continued for remedial action necessary to complete construction of intercepting sewers and pumping stations.

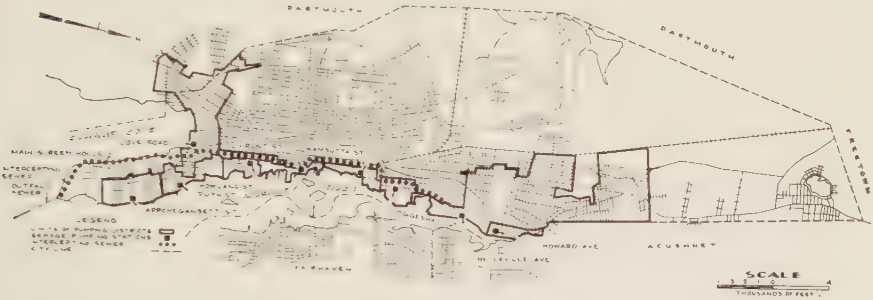


FIG. 2.—ROUTE OF SEWERS.

The city decided in 1947 to resume work. Fay, Spofford & Thorndike were retained to make engineering investigations and designs, and to supervise the construction of intercepting sewers and pumping stations in two of the districts. Separate sanitary sewers were then constructed in these two districts, together with separate sewer service connections. Roof leaders and drains were left connected to the old sewers which are now used only as drains.

We were further authorized in 1953 to report on all improvements necessary to complete the construction and to make necessary repairs and replacements of equipment for the intercepting sewer system. Only part of this report has been submitted and accepted.

The cost of constructing separate sanitary sewers throughout the remaining districts was considered too great. It was, therefore, decided to intercept and pump an amount equal to two times the dry weather rate before permitting overflow to the near-by river. Construction is now under way for an intercepting sewer and pumping station in the Howland Street pumping district, as well as for the replacement of equipment in the old pumping stations.

The general conditions of pollution existing in Acushnet River and in Clark's Cove are caused in part by adjoining towns of Fairhaven and Dartmouth. While these towns have made some preliminary investigations regarding means to abate this pollution, New Bedford is the only municipality which has appropriated large sums and gone ahead with a definite construction program.

The scope of this paper is to discuss determination of capacity of main intercepting and outfall sewers; the investigation, design, and construction of new intercepting sewers and pumping stations; new equipment for old pumping stations; sewer overflow and regulating devices; as well as screening and grit removal equipment. The methods used to gage existing flows and to establish design capacities will be described where they are different from conventional methods. We will also discuss the effects of recent hurricanes on the sewerage system and the measures being adopted to protect the older pumping stations from future hurricane and high water damage.

Investigation of Intercepting and Outfall Sewers

We have made an investigation of the main intercepting sewer and of the outfall sewer to determine if they have ample capacity to accommodate the additional discharges from the future pumping stations, as well as to receive the increased discharge within the 30-year period of our design, taken to be until the year 1985.

Gagings were made of the flows in the intercepting sewer by measuring the actual velocity, cross-sectional areas, and differences in water surface elevations at various manholes. Velocities were measured by the use of fluorescein dye injected into the sewage stream, and time of transit was measured between manholes. Difficulty was had in observing the passage of the dye until wood chips were mixed in the dye solution before discharging such solution into the sewer; then with the use of a mirror to reflect the sunlight into the manhole opening, it was possible to determine the occurrence of the dye at the next manhole opening downstream. With these data at hand, it was possible to compute the average velocity and the roughness coefficient "n" in the Manning formula for the particular stretch of sewer being gaged. The calculations of "n," based on these data, varied between 0.013 and 0.014. An "n" of 0.014 was used in the original design in 1910. Differences of elevation of water surfaces in adjacent manhole openings were then taken throughout the twenty-four hours of the day at half-hour intervals. Corresponding rates of flow were then computed.

This method of gaging was used successfully on the upper reaches of the intercepting sewer but was not effective in the sections below the main screen house where the crown of the sewer was below tide level and the sewage flow was consequently under surcharge. To make matters more difficult, manhole openings in the lower reaches of the sewer were on the side of the intercepting sewer. Stagnant water and scum in the manhole opening above the crown of the sewer did not indicate changes in the color of the sewage flow. We investigated the use of the Allen salt velocity method but found it to be impracticable for this location.

Tests were made on these lower sections of the main intercepting sewer and on the outfall sewer using fluorescein dyes, together with radioactive salts. Small centrifugal pumps (2″ diameter), with the suctions placed in the flow of the sewers at each manhole opening, were used to maintain a continuous sampling of the flow. Observation of the pump discharge in strong sunlight or artificial light would indicate at once when the color of the sewage flowing in the interceptor changed. Test tube samples were collected at 5-second intervals during the passage of the dye. Looking down at the rack of samples, against a white background, maximum color was easily determined. The range of color intensity with time recorded enabled computations to be made of center of gravity and velocity of the colored sewage.

We were assisted in our gagings of the intercepting sewer and of the outfall sewer by members of a research staff working under the direction of Dr. Harold A. Thomas, Jr., Harvard Graduate School of Engineering. The research staff desired to obtain certain data relative to the behavior of radioactive salts in sewage and sea water. The aforementioned samples of colored sewage also contained radioactive material which had been injected into the sewer at the same time as the dye. Laboratory analysis of these samples gave a very much more sensitive measurement of the velocity of the sewage than was obtained by the use of the dye. The results, however, confirmed those computed from the dye observations.

The rates of discharge of the outfall sewer were estimated by observing differences in elevation between surface of sewage in the junction house or outlet gate chamber at Fort Rodman and the corresponding elevation of tide at the outlet. The velocities and roughness coefficient for the outfall pipe were determined from velocities obtained by the use of fluorescein dye. According to these data, a roughness corresponding to an "n" of 0.023 was indicated. Additional tests will be made to determine if this high coefficient is due to something causing a partial plugging of the outfall sewer. The condition of the outfall pipe was partially determined by inspection of a diver going into both the influent and effluent ends of the outfall sewer. The diver went into the outfall sewer at each end to a distance of approximately 50 feet and reported that the outfall sewer was almost entirely free of any deposits or growths.

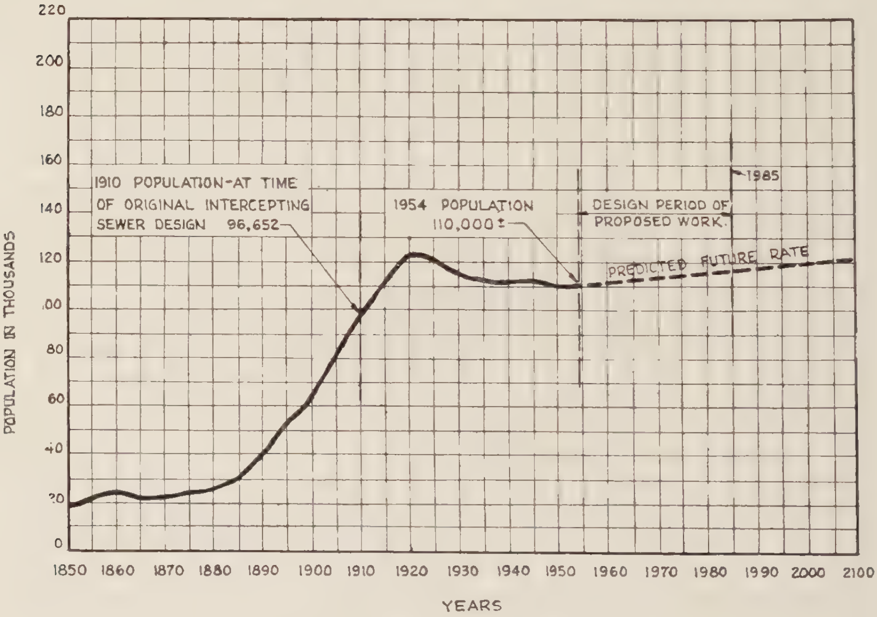


Fig. 3.—Population Curve.

The chart, Figure 3, showing growth of population, indicates that when the main intercepting sewer was designed in 1910, New Bedford's population had increased about 47 per cent in the previous 10 years. Actual population growth which has occurred since that time is also shown. The slump in population growth was due to economic conditions and the moving away of a large portion of the textile industry.

A paper published in the magazine "Civil Engineering" by Mr. John E. McLean (February 1952) cites this New Bedford population curve as a typical example of how actual population cannot be anticipated by any statistical method if a major change should take place in the economic life of a community. It is well to point out that the consulting engineers, Metcalf & Eddy, in their review, in 1910, of the City Engineer's design, stated—"that the growth of New Bedford has depended primarily upon the development of a single industry—the manufacture of cotton goods." They went on to state, "in view of the growth of this industry in different parts of the country, it may be doubted whether the recent rate of growth in New Bedford will continue for a long period of time in the future." The rate of increase originally estimated by the City Engineer was accordingly reduced from 300,000 and an estimate of population of 210,000 for the year 1940 was adopted. The intercepting sewer was designed for the aforesaid requirements. As future events dictated, the results have been to have about 100 per cent additional capacity with correspondingly unsatisfactory velocities prevailing at normal rates of flow. The average daily maximum velocity in the section of the main intercepting sewer below the screen houses is less than 1.5 feet per second.

There is excess capacity provided in the intercepting sewer as compared with the more limited capacity of the outfall sewer. This causes an interesting hydraulic condition. The action is similar to that experienced when storage tanks are provided between intercepting and outfall sewers. The rate of flow in the outfall sewer varies according to the difference in elevation between the sewage level at Fort Rodman and the elevation of tide. The rate of flow in the intercepting sewer below the screen house is at times higher than that of the outfall sewer and at times lower, depending on variations of flow in contributing sewers and head conditions prevailing in the outfall sewer. At certain times in the tidal cycle the flow in the intercepting sewer actually changes in direction, while the direction of flow in the outfall sewer continues outwards. Similar phenomena have been extensively studied and reported upon by Professor J. R. Daymond in several articles in "Journal of the Institution of Civil Engineers."

The design of the main intercepting sewer and outfall sewer contemplated the construction of two 60-inch diameter outfall sewers. When work was halted on the construction of the intercepting sewer system in the early 1920's, only one of the outfall sewers had been constructed. A sluice gate and an outlet were provided for the future construction of a second 60-inch subaqueous outfall. The expected increase in population did not materialize and, consequently, the expected sewage flows were not experienced. It is likely that the present outfall will continue to have sufficient capacity to accommodate the average maximum rate of dry weather flow until the year 1985, which was taken as the design period.

Tide gates were provided in the original construction at the junction house between the intercepting sewer and the outfall sewer. We have been told that just prior to the 1938 hurricane, these tide gates were removed and stored nearby. These tide gates were lost in the tidal wave which accompanied this 1938 hurricane and were never found. We do not consider that tide gates are needed for this installation.

Our investigation shows that the intercepting sewer, with the exception of one section, is adequate to receive the average maximum rate of dry weather flow in 1985. The exception is at the point where the intercepting sewer on Acushnet Avenue crosses Wamsutta Street. This deficiency in sewer capacity was brought about by the elimination of a railroad grade crossing and the subsequent construction of a depressed section in the intercepting sewer. As this was not contemplated in the original design, the available slope does not develop a velocity of sewage flow high enough for the proper operation of the depressed sewer. This depressed portion of the sewer is, therefore, further reduced in capacity by large accumulations of grit.

Acushnet Avenue at Wamsutta Street is congested with many underground structures, including, besides the intercepting sewer, a storm sewer of 7-foot diameter, a 30-inch water main, a large gas main, and numerous electric ducts. as well as foundations of columns and walls supporting the railroad bridge. Space is not available for widening of the intercepting sewer. Since it is not possible to increase the capacity of the sewer in the street, we are considering the possibility of constructing a by-pass of this street intersection by tunneling under the railroad tracks and crossing Wamsutta Street uphill from the Acushnet Avenue intersection, returning to the line of the intercepting sewer a few hundred feet downstream of this intersection.

The intercepting sewer and the outfall sewer provide a capacity greater than the estimated average maximum dry weather flow. The capacity in excess of this flow is available to carry a small portion of storm water flow.

About thirty sewage regulators were installed on the principal combined sewers just above their point of connection with the intercepting sewer. These were constructed at the same time as the intercepting sewer. The regulators were intended to operate so that waters from light rainfalls, in addition to sanitary sewage, would be accommodated in the intercepting sewer, but heavy showers or storms would cause the entire flow of the combined sewers to be diverted and to overflow through the old outlets. These regulators are similar to those installed in many cities where combined sewers are used. When the rate of flow exceeds a predetermined amount, the sewage should overflow through side overflow weirs to pass through an inverted siphon under the intercepting sewer and thence into the old outlet and to nearby tidal waters. The rise in elevation, so as to cause overflow when rate of flow increased, was accomplished by the throttling effect of a pipe, much smaller in diameter than the combined sewer, connecting between the sewage regulator structure and the intercepting sewer.

There was provided in these regulating structures a float-operated gate. This gate was intended to close automatically when the intercepting sewer was filled or flowing at capacity, and as soon as the flow subsided in the intercepting sewer, it was intended that the gate would open and permit the flow to resume from the combined sewer into the intercepting sewer. It was hoped by these means to prevent overloading of and damage to the intercepting sewer through connection with these combined sewers during storm flows. The combined sewers, in aggregate, during storm flows, are carrying many times the capacity of the main intercepting sewer.

During the course of our investigations, we inspected the operation of many of the sewage regulators. We found that the function of permitting dry weather flow, plus a certain amount of storm water flow into the intercepting sewer, was fairly satisfactory. The provision requiring the use of a float-operated gate, however, was found in all cases to be inoperative, as the moving parts of the apparatus became corroded in a short time, rendering the gate operation unresponsive to the elevation of the sewage level. At one regulator we found that the flow in the intercepting sewer, because of obstructions due to silt deposits, was backing up and flowing through the regulator over the overflow weir and discharging with the flow of the combined sewer into the nearby waters of Acushnet River. To remedy this condition, we removed the float-operated gate, substituting for it a simple flap gate or tide gate; we also redesigned and replaced the connecting pipe between the regulator and the intercepting sewer and rebuilt the chamber. Our computations of the throttling effect of the connecting pipe enabled us to establish a setting for height of weir planks in the overflow chamber so that when the flow exceeded three times normal average maximum dry weather flow the sewage in the overflow chamber would flow over the weirs. It was considered advisable to remove the float-operated gate because of the relatively large capacity of the intercepting sewer at this point, and also because of the need for large flushing quantities to remove silt deposits.

A large main screen house was included in the original construction as a part of the intercepting sewer system. This screen house contained three grit channels with cage screens at the entrance to each channel.

It was intended that the grit would be removed by means of sand ejectors. The grit was then to be washed by devices similar to those used in connection with slow sand water filters. As far as is known, these devices had never been used upon sewage deposits. The washing water and light organic matter were to be returned to the sewer. The grit was to have been hauled away. Provision was made in the design so that if this equipment failed to perform satisfactorily, grit could be removed by means of overhead crane and clam-shell bucket.

The grit removal system has not been operated for many years. The overhead crane and clam-shell bucket were not installed. Silt and grit deposits in the intercepting sewer downstream of the screen house have accumulated to a depth of about 3 feet.

We are proposing that the grit channels be modified to accommodate conventional endless chain drag type scrapers which will convey the grit to a hopper. It will then be agitated and washed by a pump-operated device and will be transported by grit elevators to flow by gravity through an overhead pipe to watertight bodied trucks. This material will be hauled away and buried at a suitable location.

Design of Howland Street Intercepting Sewer and Pumping Station

The designs for a new sewage pumping station and for a new intercepting sewer in Howland Street were based on the extent of contributing areas with an allowance for infiltration in the sewers, and were verified in part by records of water consumption of the larger industries in the district.

Computations of sewage quantities based on contributing areas according to their classification under the zoning ordinance are summarized as follows:

14	acres zoned "Industrial A"	@ 20,000 gal./acre	280,000 gpd
50	acres zoned "Industrial B"	@ 20,000 gal./acre	1,000,000 gpd
19	acres zoned "Business"	@ 25,000 gal./acre	475,000 gpd
2	acres zoned "Residence A"	@ 6,250 gal./acre*	12,500 gpd
0.4	acres zoned "Residence B"	@ 12,500 gal./acre**	5,000 gpd
16	acres zoned "Residence C"	@ 18,750 gal./acre***	300,000 gpd
			2,072,500 gpd

*25 persons per acre @ 250 gal./day cap.
**50 persons per acre @ 250 gal./day cap.
***75 per sons per acre @ 250 gal./day cap.

Allowance for infiltration 2.8 miles of existing sewers @ 100,000 gal./mile/day	280,000 gpd
	2,352,500 gpd
Maximum rate of sewage flow for full development of the area—say	2.4 mgd

This rate may appear unusually high for area of district. However, water consumption records as furnished by New Bedford Water Department show maximum quarterly billings for the industries located at the foot of Howland Street of about 128 million gallons. An estimated 40 per cent of this water is wasted to the sewers. This amounts to an average rate of about 750,000 gallons per day for this limited part of the contributing area.

The basis of design used for the proposed Howland Street Pumping Station provides pumping capacity to handle at least the maximum rate of sanitary sewage and infiltration expected 15 years hence, plus at least an equal allowance of storm water flows.

The layout of the pumping station is shown on Figure 4. The initial pumping installation will be two pumps with space and piping arrangement for a third pump. This third pump will be necessary in the future if additional industries having abnormal quantities of wastes should locate in the district.

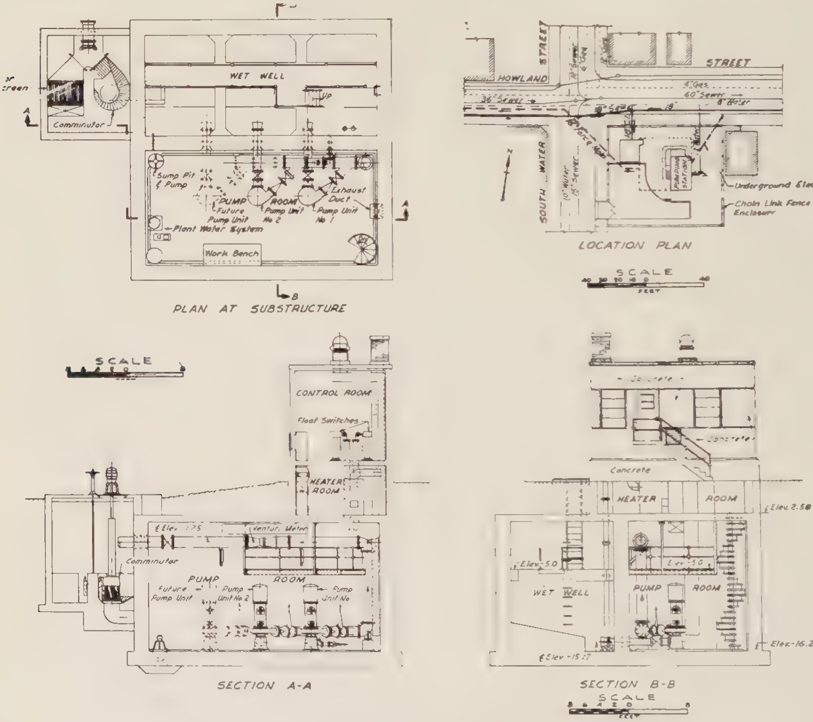


FIG. 4.—LAYOUT OF PUMPING STATION.

The initial combined pumping capacity will be 3800 gpm, or 5.5 mgd, which represents about 2.3 times the maximum rate of sewage flow, and will provide for the pumping of a considerable quantity of storm water. During heavy rainstorms, the rate of flow will exceed many times the capacity of the pumps because the contributing sewers are combined sewers. Electrical controls will be so arranged that during these periods pumps will shut off and the total sewage and storm waters will be allowed to overflow via the existing 72-inch Howland Street storm sewer into tidewater. This will help to reduce the overloading of the intercepting sewer and outfall sewers and will prevent power wastage.

Overflow structures are being built on the two principal contributing sewers at their point of connection with the new Howland Street intercepting sewer. The intercepting sewer in this district was designed according to our conventional procedures. These methods have been described in an article by Mr. R. W. Horne published in the Journal of the Boston Society of Civil Engineers of April 1943.

Close proximity of building foundations, together with unsatisfactory soil conditions, indicated by our borings, made it necessary that a portion of the new intercepting sewer be constructed inside of the old 72-inch Howland Street sewer. A new 10-inch cast iron sewer will be supported on concrete pedestals anchored to the invert of the large sewer for the length of new sewer to be placed below the center line of large sewer. Where the gradient of the new sewer causes it to be above the center line of the old sewer, the new sewer will be supported on roll hangers suspended from the top of the old sewer. This manner of supporting was adopted to prevent obstructing more than one-half of the area of the old sewer. The old sewer must remain in use to act as an overflow during storms. Existing sewer service connections from nearby industries will be extended into the interior of the old sewer to connect with the new intercepting sewer.

The new intercepting sewer in Howland Street will receive the flows from the existing sewers. The dry weather flow, plus at least an equal amount of storm water in the large outfall sewer, will be intercepted by means of an existing diversion structure. These flows will be carried to the new pumping station and will be pumped into the main intercepting sewer.

It is expected that large amounts of cloth remnants will be in the sewage because of the garment industries located in this district. A comminutor with by-pass bar screen is therefore being provided at this station.

The wet well where the sewage collects before pumping was sized so as to provide about four minutes' pumping time for any one pump. Another consideration was to provide about 8 minutes' detention time at average maximum rate of flow.

A walkway is provided in the wet well so that walls may be easily cleaned. Railings, stairways, hatch cover plates, and other exposed metal work, except for cast iron equipment, are being constructed of aluminum. This will eliminate the need for painting in these areas.

Floats, float pipes, and float switches are of a conventional type. The float switches are of the multicontact type and will provide the following series of operation:

With rising level of sewage in the wet well, the float switch will start Pump No. 1 at Elev. minus 9.5 and, if level continues to rise, it will start Pump No. 2 at Elev. minus 9.0. If the level still continues to rise, the switch will shut off both pumps at Elev. plus 1.5. At this latter elevation, which is expected to be reached during heavy rainstorms, sewage will be overflowing at the new overflow structures into the old Howland Street sewer and thereby be discharged into Acushnet River.

As the storm runoff diminishes and the level of sewage discharging over weirs in overflow structures recedes to Elev. plus 1.0, the float switch will start Pump No. 1 and as the level continues to lower the same float switch will start Pump No. 2. Both pumps will continue and when the level in the wet well reaches Elev. minus 13.5, the float switch will shut off both pumps.

Sewage pumps will be of the close-coupled type and will be located with their motors in the pump room. A sump pump is provided in the pump room to remove any accumulation of leakage or to dispose of the contents of a sewage pump whenever it becomes necessary to drain one off for cleaning.

The size of sewage pumps is controlled by our specification that the velocity in discharge nozzle shall not exceed 15 feet per second when the pump is discharging at maximum specified rate. We call for the size of valves and fittings on discharge and suction lines to be one size larger than pump nozzles, and sizes are increased along the discharge header so as to limit velocities to not more than about 8 feet per second.

A system curve of the station piping and force main was plotted to determine the total dynamic heads against which the pumps would need to operate. Head losses were computed, using coefficients for valves, Venturi meter, fittings, and piping as furnished by the various manufacturers, except that losses in laterals were computed using coefficients listed in Kaufman's "Hydromechanik."

Adjustable weight levers are attached to hinge pins of check valves to prevent troublesome slamming of check valve clappers. It has been our experience that if these are carefully adjusted and firmly set at time of pump field tests, no later troubles on this score will be had. At time of design, we investigate the possibility of water hammer occurrence in station piping and force main and, if conditions require, take necessary protective measures.

A Venturi meter is provided in the discharge piping so that daily record will be maintained of the station's operation. Recording ammeters are also provided on each motor control panel at the switchboard. Deviations from normal amperage indicate faulty operation of pumping units and we expect that these meters will indicate the promptness with which remedial action is taken by maintenance staff.

The station will be heated by means of an oil-fired steam boiler with steam coils placed in the ventilating duct.

A plant water system is being provided, together with required check valve cross connection arrangement, to ensure that water used in the pumping station cannot be back-siphoned into the city's domestic water supply. This plant water will be used for water seals on sewage pump stuffing glands, as well as for flushing water to Venturi meter.

The entrances to the wet well and to the pump room side of the station are completely separated. The threshold of the entrance to the pump room is kept above the elevation of highest known floods. The top of float well pipes is also kept above this elevation so that flood waters may not enter pump room from wet well. The records for 1938 and more recent hurricanes show an elevation as high as 10.6. We adopted a flood level elevation of 12.5 for all New Bedford sewage pumping stations built under our supervision. All elevations refer to New Bedford City Base which is 0.85 feet above mean high tide.

Renovation of Existing Pumping Stations

Pumping stations in the intercepting sewer system have been operating as long as 40 years without any substantial replacement of equipment. Pump volutes have worn through and have been patched with concrete. Electrical controls have been patched and mended. Replacement parts for most of the equipment are not available. Conditions are such that almost 100 per cent replacement is now required. New pumping equipment and electrical controls, long needed, are now being installed in these old stations.

In order to design new pumping equipment for the old stations, it was necessary to determine capacity required and the head conditions which govern design of the new pumps. We computed the probable inflow for each station in the same manner as described for the new station at Howland Street. We determined the discharge rate of existing pumps at various heads by shutting off the flow to the wet wells and pumping down a measured interval of height in a measured amount of time. The head was regulated by closing the discharge valve on the pump. We then plotted the head capacity curves for each of the existing pumps. We found that pressure gages installed on existing force mains, even with pulse dampeners, did not give results of sufficient accuracy for the high rate, low head characteristics. We, therefore, installed corporation cocks in discharge headers and used mercury manometers. By these means we were able to obtain system curves for existing discharge headers and force mains.

The next step was to determine actual discharge for these stations during a week's operation. There were no Venturi meters or other measuring devices at these stations. We installed ordinary self-starting electric clocks, with necessary transformers, on the motor leads of each pump, and by reading the elapsed time of operation every hour and the pressure readings, we were able to plot the discharge of each pump and that of the pumping station for the desired period.

Our investigations showed that new pumping units in all cases should have larger capacities than the existing units. We found it necessary also to replace with larger sizes most of the discharge pipes from the pumps because of the excessive friction head losses which would have prevailed with the old piping.

At the old stations, if the sewage level rises but a few feet above normal levels, due to blocked screens or for other reasons, the pump room and motors become flooded.

It has always been necessary to require constant attendance of operators at Cove Road and at Belleville Avenue Sewage Pumping Stations. The other stations are visited at least once each day. The reason for constant attendance at the aforementioned stations has been the need for cleaning bar screens on the influent sewers. Therefore, the screens are now being replaced by mechanically cleaned bar screen units and sewage screening grinders. Cleaning of new bar screens will be automatically controlled by means of time clock and float controls.

It is the intention to make operation of all sewage pumping stations automatic and to provide for visits by maintenance operators at least twice each 8-hour shift. Records of maintenance visits will be kept by the usual recording watchmen's clocks carried by the traveling crew. This should result in a substantial saving in operating costs of these stations.

Interesting Features of Construction

Sewer construction in congested streets was difficult due to old sewers nearby, high ground water levels, and other obstacles encountered. Well points were used extensively to combat large amounts of ground water. Old sewers which were intact until exposed would often collapse and discharge their contents into the new excavation. Many of these sewers, some several feet in diameter, had been constructed of a single thickness (4″) of brick.

As the excavation for some of the new sewers was along waterfront streets, many unusual obstacles were encountered. Old docks and careening platforms were uncovered and the timbers were found to be in excellent state of preservation. A whaling vessel, sunk in the waterfront mud for many years was encountered about 10 feet below street surface. It was necessary to use explosives to cut through the rugged hull.

Hurricanes

In common with other New England coastal communities, New Bedford experienced severe hurricanes in 1938, 1944, and in 1954. The principal damage caused by the intercepting sewer system was due to the flood waters or tidal wave which accompanied each hurricane. A tabulation prepared by the City Engineer shows a flood elevation of about 10 feet for the 1938 and 1954 (Carol) hurricanes, and about 6.7 feet for the 1944 hurricane. These elevations refer to the New Bedford City Base which is about 0.85 feet above Mean High Tide.

During the hurricane of August 31, 1954 (Carol), pumping stations at Cove Road and at Howard Avenue were flooded to a depth almost 3 feet over the entrance thresholds. Electric motors and pumps were inundated. Electric power was cut off as underground ducts became flooded. It was necessary to remove and bake out stators of motors, and to replace portions of electrical controls. Outlet structures along the shore of Acushnet River, at the many old outfalls, were torn loose and in many cases destroyed by the violent surf. Gravel was washed up into many of the outfall sewers.

New pumping stations, as previously explained, are protected from flood waters as experienced from past hurricanes by providing no outside entrance to control or pump rooms below elevation 12.5.

Old pumping stations are now being protected by closing off all openings, wherever possible, at an elevation above previous flood elevations. Lower sections of windows are being bricked up and vents are being extended to above roof level. Entrance doorways which, of necessity, must remain at present levels will be provided with steel watertight doors which can be closed and dogged tight. Electric switchboards are being mounted on raised platforms to a point above flood levels. Electric service disconnect switches are being omitted at lower elevations. Hydraulically operated cylinders are being provided for sluice gates on influent sewers so that the stations can be protected from abnormal sewage levels. We are also providing float operated devices for automatic closing of these sluice gates when sewage level approaches a flooding elevation.

With modern weather forecasting, it is expected that at least a few hours' notice will be had of the approach of a severe hurricane. Upon receipt of such notice and with above-described facilities, the maintenance crew can shut off and divert sewage flows, as well as close watertight doors at the old stations, well ahead of the storm's approach. It is necessary that, after the men have made the pumping stations secure, they get out of the area and onto higher ground until the severe storm is over.

ACKNOWLEDGMENTS

The writer has had charge of the engineering work on this project under the general supervision of Mr. Ralph W. Horne, Partner in Charge. Messrs. James A. Christenson, George C. Capelle, and William C. Traquair have served as Resident Engineers.

Construction of the two pumping districts in 1949-1950 was by Theodore Loranger & Sons-A. Piatelli Co., as a joint venture. The amount of the contract was $928,540.17.

Construction work presently under way for sewers and pumping ing station at Howland Street, and for renovations to the old pumping stations is by R. Zoppo Co., Inc. The estimated total contract price is $430,528.00.

Acknowledgment is hereby made by the writer of assistance and cooperation received from Mayor Harriman, Commissioner of Public Works Williams, Mr. R. W. Horne, and to fellow members of the staff of Fay, Spofford & Thorndike.

DISCUSSION

BY HAROLD ALLEN THOMAS, JR.,* *Member*

I SHOULD like to take this opportunity to thank Mr. Heany and his firm for making arrangements for the participation of Harvard-AEC research project group in the tracer tests on the New Bedford outfall.

The application of the Allen salt velocity test in measurement of flow rates in sewers has not heretofore been entirely satisfactory.

Common salt and other chlorides that have been used for tracers in sewers are detected by conductivity measurements or by chemical titration. Such techniques require some skill in analytic and physical chemistry and may entail bothersome calibrations when fluctuations in the concentration of domestic and industrial wastes cause rapid variations in background conductivity and chloride content. Moreover in flow systems at low Reynolds' Numbers density currents caused by the addition of brine may greatly disturb the flow pattern.

Dyes have been used with indifferent success. A considerable element of subjectivity may be introduced when under poor conditions of lighting an attempt is made to judge the flow time corresponding to the peak concentration of dye or the time half way between the first appearance and the last trace of the colored fluid. In such cases errors of 15 or 20 % in flow rates may be made.

---

*Associate Professor of Sanitary Engineering, Harvard University.

Radiotracers such as Rubidium 86 and Iodine 131 are ideal from the standpoint of sensitivity and precision. Ordinarily interference from background radioactivity is negligible. In the New Bedford tests a total of 45 millicuries of radio iodine were used. The current price of this isotope is $0.13 per millicurie. More radioactivity was used in these tests than necessary to measure the discharge since one object was to ascertain whether the tracer was suitable for measurements of tidal movements in the "slick" area at locations remote from the outfall. The results showed that these measurements were entirely feasible and that for counting the radio iodide could be separated from the sea water by a silver precipitation. The separation is necessary to eliminate interference from the dissolved solids and natural radioactivity of sea water, which amounts to about 11 disintegrations per second per liter.

The Harvard AEC research project has made extensive use of radiotracer techniques in the investigation of longitudinal mixing and "short-circuiting" in flow systems—pipes, reservoirs, tanks and rivers. The sensitivity of these methods makes it possible to observe effects much more difficult to detect than those that must be observed merely to gauge the flow. Indeed in many cases of flow measurement in medium and small sized sewers the radiotracer method is "too good" and the simple fluorescein (uranin) dye technique developed on the project and described by the author, has given excellent results in agreement with those obtained with radiotracers. It is believed that this dye technique will prove to be generally useful in evaluation of the capacity of existing sewers.

Recent advances in the theory of the salt-velocity test by Sir Geoffrey Taylor (Proc. Royal Society A *219,* 186; *220,* 446) and by the Harvard project have confirmed the validity of Prof. Allen's finding that the centroid-vertical of the time-concentration curve obtained at the downstream station may be used in estimation of flow time in the computation of mean velocity for the ordinary discharge rating of pipes and narrow open channels. Errors are usually less than 2%. In tanks or lakes or under unusual circumstances in pipes, the centroid vertical may not be the proper point for measurement of time. In these cases, however, the new theory yields suitable computational procedures for locating the point corresponding to the average flow rate. The theory has especially valuable application in investigations of the efficacy of natural purification processes that determine the capacity of streams in handling low and intermediate level radioactive wastes that may be discharged in connection with the operation of power nuclear reactors.

DISCUSSION

BY ROBERT T. JONES*

MR. HEANEY has given us a fine example and cross-section of the many diverse problems confronting the engineer in designing sewerage works and has also shown us the extent to which an engineer must improvise in completing such an investigation and design.

It was particularly interesting to note the remarks relative to "anticipating population change". These point out again the need to study the overall patient, including the economic life of the community, rather than the particular illness at the time. Even the most careful study, as pointed out, cannot anticipate population growth if a major change should take place.

Relative to design of intercepting sewers, there was published last year by this Society a paper entitled "Sewer Capacity Design Practice" by William E. Stanley and Warren Kaufman in which it was pointed out that the use of *unit per acre sewer capacity quantities* is a very useful procedure as a basis of checking estimates of sewage flow rates. Using these authors' procedure, it will be found that the design for the Howland Street intercepting sewer falls very nicely in the middle of the chart and lies close to this unit rate found for the City of Lynn. This is about 24,000 G.P.A.D.

In respect to the investigation of sewage flows by determining the hydraulic gradient through a specific pipe section, it would be interesting to know, in general, how the roughness coefficients came out; that is, were they reasonably close to the usual accepted values for such pipe lines. In a past study by Whitman & Howard of circumstances similar to which Mr, Heaney speaks, it was found that the roughness coefficient of clay tile pipe was quite close to the generally accepted "C" values in the Williams and Hazen formulae; (that is, "C" = 110).

---

*Engineer, Whitman & Howard, Engineers. Boston, Massachusetts.

The only further comment I would like to make, or I should say "question", and this was brought to my mind by Mr. Heaney's remarks about the condition of the existing stations and by my own experience with such stations, concerns the design of dry pit type sewage pumping stations. When using close-coupled electric motor driven pumping units, the motors often are located below a potential flooding point. By this I mean a flooding in the event a pump casing or pipe breaks. I would be interested to know what Mr. Heaney or other members here think about this flooding potential and any suggested steps to avoid this.

SANITARY ENGINEERING IN ALASKA

By Clarence I. Sterling*

[Presented at a meeting of the Boston Society of Civil Engineers, held on March 2, 1955.]

Introduction

It is difficult for the uninitiated to know or to understand Alaska. Nothing in Alaska can be isolated and discussed as a separate entity. The Territory is diverse. each phase of its life and economy closely inter-related and each dependent upon the other. The problems of sanitation and the methods to be used by the Sanitary Engineer to solve these problems are varied. The Territory, as a whole, is currently a military economy. The country is underpopulated and underdeveloped and it is an example of what happens when a land is dependent upon the Federal Government for its economic and political life. Over 99 per cent of the land is owned by the Federal Government. Thus, one might say that the resources of the Territory are in "cold storage" insofar as development is concerned. On the other hand, many Federal officials and employees, as well as citizens of the Territory, have devoted their lives to making a better Alaska. It is understandable, however, that members of Congress, hard pressed by their own voting constituents, should fail to give top priority to the solving of problems of non-voting Alaskans thousands of miles away.

Historically, the Russians laid claim to Alaska and the Pacific Northwest as a result of its discovery in July, 1741, by a group of Russians led by a Danish sea captain, Vitus Bering. This party was in reality trying to find the Northwest Passage in reverse. The Russians proceeded to exploit the Territory for its furs until it was purchased by the United States in October, 1867, for the sum of $7,200,000.

As far as culture was concerned, the Russians did bring in the Russian Orthodox Church and today that continues to be the religion of a large portion of the native population.

It wasn't until gold was discovered in the Klondike in the latter part of the 19th Century that Alaska was rediscovered. When the Gold Rush was over, so tragically unsuccessful to the vast majority of the prospectors, Alaska was not considered of any great importance until the Japanese attack on Pearl Harbor in 1941. As late as 1939, there was only one military installation in existence in the Territory of Alaska and that was a post of 200 Americans located near Haines at the head of the Inside Passage. World War II brought about a tremendous change in Alaskan life and this change has continued to grow ever since.

---

*Chief Sanitary Engineer, Massachusetts Department of Public Health.

Geography and Climate

Physically, Alaska is both beautiful and monotonous. It varies from the endless "Tundra" plains of the Arctic Slope to the magnificent mountain country with its large virgin forests along the southeastern coast. It has an area of approximately 586,400 square miles, about 1/5 of the United States, and a coastline of 34,000 miles.

Basically, it is divided into three areas which are completely different from each other—the Southeast Coastal Area, the Central Plateau Region and the Arctic Slope Region.

Insofar as climate is concerned, it is a land of contrast. Temperatures at Fort Yukon in the Central Plateau Areas have ranged from 100° above zero in the summer to —78° F. in the winter. Similar variations are found in Fairbanks, a thriving metropolis, with a maximum temperature recorded as 99° above zero to a minimum of 66° below zero. As a matter of fact, when the writer was in Fairbanks last July the temperature was 90°.

Because of the low temperature, a large portion of the Territory of Alaska contains permanently frozen ground called "Permafrost". It is the existence of this permanently frozen ground that has resulted in engineering problems that are peculiar to the region. This is true not only in the field of Santary Engineering but in all phases of engineering.

In the same way, rainfall varies greatly in this land of contrasts. Along the Southeasterly coast some areas have up to 180 inches per year. In the Central Plateau, rainfall varies from 12 to 15 inches per year and in the Arctic Slope Region it is less than 5 inches.

Some of the temperature and rainfall variations in a few of the larger cities and towns are shown in the following table:

TABLE I

	Aver. Temperature *Jan.*	*July*	*Min. Temperature*	*Annual Rainfall*
Ketchikan	33°	58°		151″
Sitka	32°	55°		87″
Juneau	28°	57°	—15	83″
Anchorage	11°	57°	—36	14″
Kodiak	30°	54°		62″
Nome	— 3°	50°	—47	18″
Fairbanks	—12°	60°	—66	12″
Barrows	—17°	40°	—56	5″

POPULATION

In the 1950 Federal Census, Alaska had a population of 128,643 persons. However it has been increasing so rapidly that by 1953 it was estimated that the population was in excess of 205,000, including military personnel. Ethnically speaking the 1950 population was made up as follows:

Eskimo	15,882	
Indian	14,089	
Aleut	3,892	
Non-Whites	1,972	(Negroes, Japanese, Filipino, etc.)
Whites	92,808	

In the native population, the number of males and females is equally divided, while in the white population males outnumber the females by 186 males per 100 females. The population is concentrated in about three areas. In southeastern Alaska 28,200 persons live in an area of 34,400 square miles and here we find a considerable Indian population similar to our Indians of the northwest United States. Within a radius of 75 miles of Anchorage there are 38,500 persons residing in about 18,000 square miles. This, one might say, is largely a white populated area. Within a radius of 20 miles of Fairbanks we find about 18,000 persons residing. Here again we find a prominently white population. The remaining 1/3 of the population, approximately 44,000 persons, is spread over 500,000 square miles living in villages often containing less than 100 persons. Here we find the interior Indian, the Athabaskan, in the Central Plateau: the Aleut in the Aleutian Islands and the Eskimo in the Tundra and Central Plateau areas.

Enteric Disease in Alaska

While sanitation needs are apparent to even the most casual observer, the effect upon health is difficult to measure statistically. The incidence of enteric diseases is probably one of the best indicators of poor sanitation. Unfortunately enteric diseases are not diagnosed because of the lack of medical personnel and are not reported when diagnosed. Thus, statistically they do not appear to be the problem they actually are.

A survey made in four Alaskan villages of the causes of illness showed that intestinal upsets were in the magnitude of 18.9 per thousand people per month, compared with a similar survey made in a health district in Pittsburg of 2.3.

The best material on the subject of Enteric Disease in Alaska is a paper that was presented at the Third Alaskan Science Conference in September, 1952, by Frank P. Pauls, Director of the Regional Laboratory of the Alaska Department of Health in Anchorage.

Sanitary Engineering in Alaska

Since the beginning of World War II, when Alaska became so important to the United States for military purposes, a lot of study has gone into solving the problems of living in a frozen territory. A great amount of research has been done by the military forces as well as by civilian agencies and a great deal more needs to be done.

I will not try to go into the details of the problems other than sanitation as a considerable amount of material has been published on the subject in various technical journals.

Public Water Supplies

A public water supply is defined by the Alaska Department of Health as one that supplies 25 or more people with water. However, an analysis of the statistics on the basis that there is a pipe distribution system, the water is sold, or used on an interstate carrier, shows that there are only 34 systems serving municipalities having a population in excess of 100 persons.

Only six of these systems could be considered satisfactory at the present time. Four or five additional systems would be capable of delivering satisfactory water if they were properly operated.

In regard to water treatment, only the city of Fairbanks has a water filtration plant and this is primarily for iron removal. Eighteen of the other supplies are chlorinated while Anchorage, the largest city in Alaska, not only chlorinates the water supply received from Ship Creek but started a program of fluoridation in May, 1954. Statistics relative to the various public water supplies are shown in Appendix A.

Water for semi-public and private supplies is obtained from a number of sources. In some cases shallow wells and in others deep drilled wells are used. Deep drilled wells penetrate below permafrost to obtain water. In most cases where permafrost is penetrated the water apparently contains a large amount of organic matter often with objectionable quantities of iron and manganese. At some points where permafrost has been penetrated by deep wells along the coastal areas the water has been highly mineralized containing in one case as high as 86,000 parts per million of chlorides, making it unsuitable for water supply purposes.

In many of the other communities that do not have public systems, water is taken from either wells or rivers and is distributed by means of tank trucks. In some cases the operators of these tank truck services claim they chlorinate the water. However, the effectiveness of this chlorination practice is questionable.

There are about 16 bottling water and beverage plants in operation at the present time. Sample bottles are sent by the laboratories of the Alaska Department of Health to all known public water supply systems so that the bacterial samples are collected and submitted to the laboratory at regular intervals. An effort is made also to have an inspection of each water supply system at least once a year by a Sanitary Engineer or a Sanitarian of the Department of Health.

Present legislation is adequate to safeguard water supply systems. However, the lack of personnel makes it almost impossible to supervise them satisfactorily. As a matter of fact, legislation even permits the Alaska Department of Health to order objectionable conditions corrected or necessary works installed. However, since only 28 communities are incorporated, enforcement of that type of legislation is very difficult. Furthermore, it is not necessary to have basic legislation to establish a public water system. After a water system is established there is nothing to prevent the owners of the system from discontinuing services if they feel that the Alaska Department of Health is making too many demands upon them for needed improvements. Therefore, legislation should be enacted requiring the approval of the sources of supply, methods of treatment, etc., by the Alaska Department of Health for the establishment of municipal supplies and water companies. Once incorporated or established such systems would become public utilities and could not discontinue services at will if the best interests of the public were not served.

Public Sewerage Systems

Sewerage systems have been installed in about 15 communities, the sewage collected therein being discharged directly into rivers or tidal waters without treatment as there are no municipal sewage treatment plants at the present time. There are, however, several small sewage treatment plants that have been constructed for housing subdivisions and the major Army and Air Force installations have treatment works. These latter works are not always competently operated and as a result large quantities of improperly treated sewage are discharged into streams and tidal waters.

The majority of the outfall sewers discharging into the tidal waters terminate near the shoreline. Since the tidal range is approximately 30 feet in some localities objectionable conditions occur during the low tide when sewage is deposited upon open tidal flats.

Industrial Waste Disposal

The most important industrial waste disposal problem in the Territory is from the fish canneries located along the coastal areas. This industry cans salmon, halibut, herring, crab, shrimp and shellfish. The plants are usually located along the beaches or constructed on pilings on the edge of tidal estuaries. Due to the curtailment of the fishing season only about 90 of the 200 existing canneries were in operation in 1954. At the plants it is customary to discharge the gurry and the washings from equipment and floors together with the sewage and other wastes directly into the tidal waters. Due to the wide tidal range this frequently causes objectionable conditions along the exposed flats.

A new industry, the manufacture of paper pulp, has recently been started. A mill has been established on Ward's Cove immediately north of Ketchikan where the magnesium bisulphite method of paper pulp manufacture is being used. This modern mill cost approximately $54,000,000. In order to reduce the amount of industrial waste being discharged into Ward's Cove a large portion of these wastes is evaporated and burned as fuel with recovery of some of the chemicals. Additional pulp mills are being considered in the Sitka, Juneau and Petersburg areas at the present time.

Water pollution control studies have been made, principally at Ward's Cove in the Ketchikan area, but these activities have been stopped since no appropriations are being made to continue the work.

GOVERNMENTAL AID FOR WATER SUPPLY AND SEWERAGE DEVELOPMENT

The Alaska Public Works Act authorized the Department of the Interior to make available $70,000,000 for the construction of public works in Alaska. This money can be used for the development of water supply and sewerage systems when dollar for dollar matching funds are made available by the local community. To date approximately $30,000,000 has been appropriated for all types of public works under the original authorization. In addition to the matching fund provision, the Federal Government will buy the public works bonds issued by the municipality at 2% interest.

The difficulty in the utilization of this money is due in part to the fact that there are only 28 incorporated municipalities in Alaska and many of the improvements are needed by the communities which are unincorporated and therefore do not have the financial ability to underwrite their portion of the loan.

It was further noted that in certain cases Government agencies, either territorial or Federal, were establishing water supply systems without the approval of the source of supply or the distribution system by the Alaska Department of Health. It is a customary practice for Federal Government Agencies in the States to submit their plans to the proper authorities, namely the State Department of Public Health, for approval prior to installation. This approval of plans is particularly essential in Alaska since many Federal Agencies have constructed sanitary works and operated them for a period. Later the Agencies abandoned the responsibility for operation but the water and sewerage systems continued in operation for the service of the remaining population. Thus, there is the creation of unapproved water supply systems over which the Alaska Department of Health has little authority at a later date to correct unsanitary practices.

Sanitation in Permafrost Area

The problem of housing, water supply, sewage and waste disposal is particularly critical in the low temperature areas known as the permafrost region. In approximately 60 per cent of the total area of the Territory, or 370,000 square miles, the ground remains frozen throughout the entire year at varying depths. This permafrost may extend from within a few feet of the surface of the ground to a depth of up to 1,000 feet or more.

The depth at which permafrost starts below the surface of the ground depends to a large extent upon the type of cover material. In areas where the ground cover is thick with trees and moss it is probably only a few feet below the surface during the summer months and in areas containing relatively little vegetation the permafrost recedes to a greater depth. However, during the winter months there is a seasonal frost penetration from the surface of the ground which may either make a direct contact with the permafrost level or permit a non-frozen layer between the frozen strata. For example, in the Fairbanks area the seasonal frost has frozen water in mains 10 feet below the surface of the ground although that is a rare occurrence.

With permafrost in the ground it becomes difficult to construct stable foundations for buildings and various methods of foundation construction are being explored. The melting of the permafrost by heat losses from buildings causes unstable conditions so that many of the buildings are built upon wooden piles that have been inserted into holes drilled into permafrost. In one case refrigeration has been used to stabilize the soil under a foundation.

The obtaining of water supply becomes a serious problem, especially locating suitable sources and obtaining ground water from wells where the frost extends to any considerable depth. Wells have been drilled successfully through this material. However, the drilling process is rather difficult and expensive since the temperature of the ground is constantly below freezing, making it necessary to heat with steam or some other means any liquid in the drill hole.

The same difficulty arises in the disposal of sewage. When sewers are located in the permafrost area there is a danger that they will freeze or that the higher temperature of the sewage in the pipes will cause unstable soil conditions and thus cause failure in the sewer system.

The Sanitary Engineers at the Arctic Health Research Center have been making studies in the city of Fairbanks on the design of sewers. Figure 1 shows the results of a year's study of the temperature of the sewage, ground and air at one of the observation points. As a result of the study the sewerage system in Fairbanks is being modified in order to eliminate short lateral sewers which are liable to freeze easily.

The Army, in the construction of their installations, has utilized the so-called "Utilidor", which is simply an underground tunnel in which the sewer pipe, water mains, steam lines, electric and other utility conduits are placed, thus preventing the freezing up of the water and sewer pipes. However, the cost of the "Utilidor" is very great. While undoubtedly justifiable at a major Army installation or institution, it is not economically justifiable in the normal communities.

The problem of providing a satisfactory water distribution system in the permafrost area is a great one. However, the new water supply system for the city of Fairbanks is a fine example of what can be done as a result of extensive research and careful planning.* As a result of studies made by the engineers at the Arctic Health Research Center of the Public Health Service it was found that a normal water distribution system could be utilized using a single main in the street but with continuous flow dual service pipes to each house. It was found that, if the water in the mains and service pipes were maintained at a velocity of not less than 2 feet per second by circulating pumps placed upon the distribution system, water could be distributed without freezing in the mains. The water from the city of Fairbanks is obtained from two gravel-packed wells through permafrost located near the Nenana River. The water pumped from the wells enters a filtration plant for iron removal, is chlorinated, and then pumped into the distribution system. During certain times of the year when the temperature of the water is near the freezing point it is possible to add heat to the water through a heat exchanger from an adjacent power plant. While in the design of the Fairbanks water system underground storage tanks were placed at various points upon the system for fire protection purposes, it seemed a little questionable and costly to use such tanks in place of elevated storage tanks. As

---

*A description of the Fairbanks water supply system was published in *Public Works Magazine* in January, 1954.

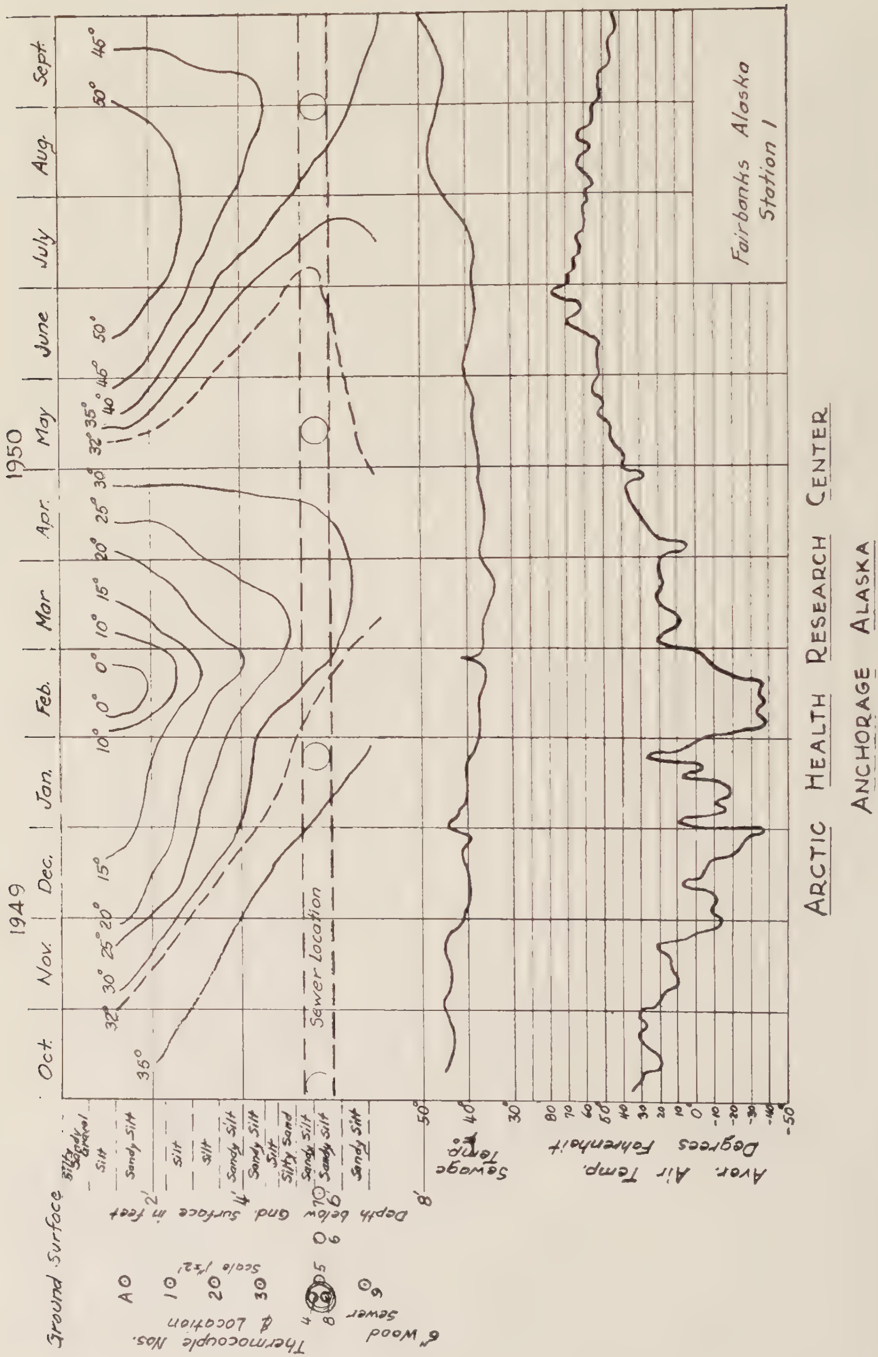

1949
1950
Oct.
Nov.
Dec.
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
35°
32°
30°
25°
20°
15°
10°
0°
40°
45°
50°
Sewer Location
Ground Surface
Silty Sandy Gravel
Silt
Sandy Silt
Silt
Silt
Sandy Silt
Sandy Silt
Silt
Silty Sand
Sandy Silt
Sandy Silt
Sandy Silt
2'
4'
6'
8'
Depth below Gnd. Surface in feet
Thermocouple Nos.
℄ Location
Scale 1"=2'
A
10
20
30
4
5
6
8
9
6" Wood Sewer
50°
40°
30°
Sewage Temp.
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
Avor. Air Temp. Degrees Fahrenheit
Fairbanks Alaska Station 1
ARCTIC HEALTH RESEARCH CENTER
ANCHORAGE ALASKA

FIG. 1.

a matter of fact, there is an elevated storage tank located at the Alaska Railroad yards within ½ mile of the muncipal water filtration plant which is successfully operated without freezing.

Sanitary problems in the Tundra area occupied by the Eskimos along the westerly, northwesterly and northerly sections of Alaska are the most critical. The Eskimos are classified as natives with health and welfare facilities provided by the Alaska Native Service, the Alaska branch of the Indian Service of the Department of Interior. The problem of the Eskimo is considerably different from that of the Indian in the States who lives on reservations. The Eskimo is an integral part of the community population enjoying full citizenship with the power of voting and the responsibility of paying taxes. He may live in villages along the coast or along the large Arctic rivers such as the Yukon and the Kuskokwim. These villages are small, usually without water supply or sanitary facilities. Water may be obtained directly from the river, collected in cisterns from roof water or obtained from the melted snow and ice. Due to permafrost, toilet facilities are lacking. It is common practice to dispose of human wastes into 50 gallon oil drums and when they are full to cart them out upon the Tundra or throw them into the sea or river.

Due to the scarcity of lumber in many of the areas houses are small, often underground and partly covered with earth in order to conserve fuel. The amount of living area is very small per person. Thus, the Eskimo has three major problems in the field of Sanitation, namely the difficulty in securing a safe water supply, the lack of proper sewage disposal measures and housing conditions that are not conducive to healthful living.

Usually there is a schoolhouse in the village either operated by the Territory Department of Education or by the Alaska Native Service. However, many of the schoolhouses suffer the same lack of sanitary facilities as the private home.

The Eskimos residing along the coastal area during the fishing season frequently move en masse from their villages to establish fishing camps upon one of the major rivers in order to catch salmon and dry it for their own use and the use of the dog teams. The families move with most of their possessions in their open boats which are usually equipped with outboard motors. The sanitary conditions at some of the fishing camps are poor, but in some cases they are better than those in the original villages. It was interesting to note that in some of the fishing camps the women had transported their gasoline-operated washing machines with them.

BETHEL—A TYPICAL COMMUNITY IN PERMAFROST AREA

Bethel is located in the permafrost area on the north shore of the Kuskokwim River some 80 miles above its mouth. It has a commercial airport as well as an auxiliary landing strip for smaller planes and is a deep water seaport for ocean-going boats.

Bethel has a population of 651 and is headquarters for an area of about 80,000 square miles. The Alaska Native Service maintains a modern 69-bed regional hospital which was recently opened. The Territory Department of Education maintains a combined high school and grammar school, which is the only high school in the area.

The water supply for this town is distributed by a private operator by means of a tank truck. The water is pumped from the river into a wooden elevated tank at a point located about ½ mile downstream from the village. From the wooden elevated tank the water flows into the tank truck and is then peddled to the residents. The water is discharged from the tank truck through a hose into 50 gallon drums (the cost of water in $1.50 per drum). This water is used by all the residents, at the schoolhouse, the National Guard Armory and was formerly used at the Alaska Native Service Hospital.

Garbage and human wastes from the village are thrown into the river and flow downstream toward the water intake. In addition, the river has encroached upon the banks causing some of the bodies in the cemetery to be carried downsteam and on one occasion a report was circulated that the intake to the pump was stopped by an arm. While we are not so concerned about the danger of disease from the dead as from the living, this is not a very pretty picture. The operator of this water service states that he puts a few drops of "Chlorox" in the tank on each trip but that is debatable. The service is maintained during the winter months when the pump is moved out upon the ice in the river which is about four or five feet thick and water can be taken from a point nearer the town.

When the new hospital was built a well was drilled some 467 feet deep at the hospital and water was successfully obtained from below permafrost. Permafrost at that point was 427 feet in depth. The water from the well is pumped into storage tanks, then through sand pressure filters, then chlorinated and a satisfactory water supply is now available for the hospital.

One other person in town, an airline pilot, has drilled a well about one mile from the hospital where permafrost was only slightly over 100 feet in depth and this individual is just waiting for the installation of a new electric generator to have a ground-water supply for his own house. Across the river at the commercial airport it was stated that there was no permafrost in that immediate area.

Bethel is an unincorporated village with few financial resources, especially among the native population. The Government agencies could improve the situation by providing a safe water supply at their own installations. For example, a well could be installed by the Territory School Department at the high school, or the Alaskan National Guard could install a well at their armory. The danger of drinking contaminated water would then be reduced if such wells were used as the source of water for the water cart service even if a distribution piping system was not installed. The Government Agencies have a definite responsibility to include a safe water supply and adequate sewage disposal facilities in their plans wherever buildings are constructed. Obviously it is important to the Alaskan National Guard that their National guardsmen, who come from the village of Bethel and vicinity, be healthy and free from enteric disease if they are to be utilized in the defense of Alaska.

Arctic Health Research Center

There are several Federal Agencies carrying on research in regard to the solution of specific sanitation problems which are complicated by low temperature and permafrost. One of the most important of these research groups is the Arctic Health Research Center of the U. S. Public Health Service. This Center is located in Anchorage although its activities are carried on in various parts of Alaska. It was created in 1948 to conduct a program of arctic scientific research and to investigate health and sanitation conditions in Alaska.

It is through this station that technical assistance is rendered to the Alaska Department of Health. As an example, the Center assigned three Sanitary Engineers to staff the regional offices of the Division of Sanitation and Engineering.

The writer in his visit to the research station was particularly interested in research on environmental problems. One of the principal reasons the station was established was to find solutions to arctic sanitation problems, particularly how water supplies and sewage facilities could be developed in permafrost areas. The sanitation staff of the station consists of three sanitary engineers, a bacteriologist, an aquatic biologist, a chemist, a sanitarian (who is used principally for the preparation of research center reports) and a laboratory helper. One of the sanitary engineers has been assigned to the Alaska Department of Health office in Fairbanks as the Research Center in Anchorage is not located in a permafrost area.

Most of the sanitation staff have been working on Alaskan problems for four or five years and appeared to be very competent in their field. One of the most valuable pieces of work done by the staff was the development of a method of water recirculation to prevent freezing in the water distribution systems. This was given a practical application by the installation of the new Fairbanks water system.

Research has been conducted on the prevention of the freezing of sewage and sewerage systems, on the practicability of utilizing septic tanks and subsurface disposal works for small installations in permafrost areas and on the penetration of cold to determine the cause of freezing in sewers. Some research was also done on well construction and the prevention of freezing of water in wells located in permafrost.

Extensive studies were made using chromogens as pollution indicators to determine the rate of travel in ground-water and the effect upon the quality of water in nearby wells. This work, however, was not done in a permafrost area.

ARMY ENGINEERS PROJECT "PERMAFROST"

The Army Engineers Corps has a project called "Permafrost" located some five or six miles north of Fairbanks. Here studies are made upon the use of materials in airplane runways in a permafrost area, the effect of various types of soil cover upon the depth of permafrost below ground surface, methods of preventing freezing of water in wells drilled through permafrost, primarily through the use of compressed air, and the study of concrete and hollow tile foundation construction.

At the time of the writer's visit possibility of undertaking a project on the study and effect of sewage and sewage flow in permafrost was being considered.

Again it appeared that if an active liaison could be maintained between the staff working at the various research centers undoubtedly the progress of the research program would be accelerated. Thought should be given to development of projects as joint ventures. For example, as stated before, the Arctic Research Health Center has been conducting studies on sewers, sewage disposal, etc., in permafrost in Fairbanks and at the University of Alaska, both areas being located within five miles of the Army "Permafrost" project. A joint venture in this project might be profitable at this time as the Army Engineers seem to have more funds available and the Arctic Research Center has professional personnel but inadequate funds.

Sanitation Problems Requiring Additional Research

There are numerous problems in the field of sanitation that require additional research. This same situation prevails in other parts of the world. However, specific problems relating to Alaska should be given an early priority. A few suggestions for needed projects are as follows:

(1) Further study on sewage disposal for private homes and small installations by the use of a septic tank with subsurface disposal of the effluent in permafrost areas.

(2) Research in regard to the proper slope for laying sewers and house connections in permafrost areas.

(3) A study of community sewage disposal in permafrost areas by the use of the lagoon method. The writer feels that during the winter months such lagoons would be ice covered and during the summer months with the long daylight hours biological growth would be very prolific resulting in reasonably good purification of the sewage.

(4) Research in regard to the use of sanitary landfill for the garbage and refuse disposal to determine if it is necessary to cover the material during the extremely cold seasons of the year.

(5) Study to determine the area required per person for healthful housing in relation to fuel consumption and body comfort.

(6) Study of the development of an aluminum housing unit.

There are approximately 8,000 Eskimos living in areas without wood being available readily for house construction, which would indicate that some other material should be developed that would be durable if not more economical to use.

(7) A study of method of heating the homes of the Eskimo more effectively than present methods. For example, placing the heating unit below the floor level for better utilization of the heat.

(8) A study of methods of sewage disposal in the arctic area for individual homes not having running water available.

APPENDIX A

Public Water Supplies in Communities with Population in Excess of 100

Community	1950 Population	Ownership	Source	Extent of Distribution System	Treatment	Comments on Sanitary Status
Anchorage	30,000*	City	S	City fairly well covered	Cl., F.	Requires additional treatment and supply
Angoon	429	Community	S	Limited	None	Requires Cl.
Annette Village	302	CAA	S		Cl.	Satisfactory
Bartlett	150	Alaska Gov.	—	Sufficient	—	Takes Water from Seward
Chilkoot Barracks	125	Private	S	Adequate	Cl.	Questionable Need replacements
Cordova	1,165	City	S	Limited	None	Rehabilitation required
Craig	374	City	S & G	Limited	None	Unsatisfactory
Douglas	699	City	S	Adequate	Cl.	Questionable
Dutch Harbor	173	Abandoned Gov.	S	Adequate	Cl.	No responsible operat.
Fairbanks	18,000*	City	G	Limited**	Cl. Iron Removal	Satisfactory. Expansion program in process
Haines	338	City	S	Adequate	Cl.	Unsatisfactory operation
Hoonah	563	City	S	Adequate	Cl.	Unsatisfactory operation
Hydaburg	353	City	S	Limited	Cl.	Unsatisfactory operation
Juneau	5,956	City	S & G	Adequate	Cl.	Satisfactory
Kake	376	City	S	Limited	None	Unsatisfactory
Ketchikan	5,305	City	S	Adequate	None	Requires treatment
Klawock	404	City	S	Limited	None	Inadequate sources
Kodiak	1,710	City	S	Adequate	Cl.	Requires additional treatment

*Estimated population as area of city has been enlarged since 1950 Census.

**Serves about 3.000 persons.

APPENDIX A—*Continued*

Community	1950 Population	Ownership	Source	Extent of Distribution System	Treatment	Comments on Sanitary Status
Metlakatla	817	City	S	Limited	None	Requires treatment
Mountain Point	109	Community	S	Limited	None	Unsatisfactory
Nenana	242	Alaska RR	G	Extremely Limited	Cl.	Unsatisfactory
Nome	1,876	Mining Co.	G	Extremely Limited	None	Summer supply unsatisfactory
Palmer	890	City	G	Adequate	Cl.	Satisfactory
Pelican City	180	Private	S	Limited	None	Unsatisfactory
Petersburg	1,619	City	S	Limited	Cl.	Unsatisfactory oper.
St. George Island	187	Department of Interior	S	Limited		Unsatisfactory oper. Need better source
St. Paul Island	359	Department of Interior	S	Limited	Cl.	Need better source
Saxman	167	Community	S	Limited	None	Unsatisfactory
Seldovia	437	Community	S	Extremely Limited	None	Unsatisfactory
Seward	2,114	City	S	Limited	None	Treatment plans in process
Sitka*	3,132	City	S	Adequate	Cl.	Satisfactory
Skagway	758	City	S	Adequate	Cl.	Questionable operation
Valdez	554	City	G	Adequate	Cl.	Satisfactory
Wrangell	1,263	City	S	Limited	Cl.	Questionable operation

S—Surface Water; G—Ground Water; Cl.—Chlorination; F—Fluoridation.

---

*Includes Mt. Edgecombe, Population 1,147.

APPENDIX B

Bibliography

The following is a list of references which contain valuable material in regard to the sanitary conditions and the problems of sanitation in Alaska. Many of the references noted contained extensive bibliographies which will not be included in those listed here.

1. Pauls, Frank P. Enteric Diseases in Alaska.
2. Alter, Amos J. Relationships of Permafrost to Environmental Sanitation.
3. Alter, Amos J. Water Supply Problems in Low Temperature Areas.
4. Alter, Amos J. Arctic Sanitary Engineering. Federal Housing Administration. 1950.
5. Whitaker, Harold A. A Proposed Program of Research in Environmental Sanitation in Alaska. 1949.
6. Alter, Amos J. Water Supply in Alaska Journal American Water Works Association. June 1950.
7. Branton, C. I., and Fahnestock, C. R. Use of Native Alaskan Materials for Farm and Home Construction. Bulletin 17, Alaska Experiment Station, November, 1953.
8. Anonymous. Community Facilities in Alaska. Present Status and Recommendations. Alaska Department of Health. 1949.
9. Chambers, John S., and Magnusson, Harris W. Seasonal Variations in Toxicity of Butter Clam from Selected Alaska Beaches. Special Scientific Report: Fisheries No. 53. Fish & Wildlife Service, U. S. Dept. of Interior.
10. Chambers, Craven and Galerman. Technical Studies on the Alaska Butter Clam. Technical Report No. 3, Fishery Products Laboratory, Fisheries Experimental Commission of Alaska, Ketchikan, August, 1952.
11. Magnusson, H. W., and Carlson, C. J. Technological Studies on the Alaska Butter Clam—Review of Problem of Occurrence of a Toxin. Technical Report No. 2, Fishery Products Laboratory, Fisheries Experimental Commission of Alaska, Ketchikan, September, 1951.
12. Anonymous. Public Health Problems In Alaska. Public Health Reports, Vol. 66, No. 29, July 20, 1951. Federal Security Agency, Public Health Service.
13. Page, William B. Design of Water Distribution Systems for Service in Arctic Regions. Water and Sewage Works, August, 1954.
14. The Alaska Health Survey Team. Alaska's Health: A Survey Report. Report to United States Department of the Interior, 1954.

In addition, the reports of the various subcommittees of the Division of Medical Science of the National Research Council on various arctic sanitation studies were reviewed as well as the biennial reports of the Alaska Department of Health and the latest biennial report of the Alaska Development Board entitled "Charting Alaska's Progress".

GRAPHICAL SOLUTION FOR TORSIONAL PROBLEMS HYDRODYNAMICAL ANALOGY

By Alexander Wargon*

Synopsis

While torsional problems are very common in engineering, the lack of a general, elementary solution becomes a very serious handicap in practical engineering.

Solutions are available in the literature for particular prismatic cross-sections, such as rectangular, circular, triangular and thin walled profiles, but no general solution for an arbitrary section is known.

However, for an arbitrary section, the membrane analogy as a "model" of stresses has been found very useful (7, 8). An apparatus for the measuring of the lateral deflection of the membrane is required.

The author presents in this paper a practical solution to the torsional problem, by a graphical method, which requires no special apparatus and whose degree of accuracy is, probably, of the same order of magnitude as in the soap-film membrane method.

Introduction

In general, the torsional problem of twisted bar of a constant, but arbitrary cross-section, is a three-dimensional problem.

Saint-Venant (2) made an assumption with regards to certain stress components and found that the problem may be reduced to a plane one.**

Solution of the two dimensional problem may be reduced finally to an integration of a differential equation, which, after introducing the so-called stress-function becomes

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = -2G\theta \qquad (1)$$

Function ϕ is defined by $\tau_{xz} = \frac{\partial \phi}{\partial y}$; $\tau_{yz} = -\frac{\partial \phi}{\partial x}$

The torque $M_T = 2\iint \phi \, dx\, dy$ (2)

---

*Structural Engineer, Linenthal & Becker, Engineers, Boston, Mass.

**Assuming that there are no body forces.

While no general solution or expression for ϕ function is available, equation (1) also defines laminar flow, deflection of a membrane loaded by uniformly distributed load and equal axial load, and has also an electrical analogy.

This paper will be concerned with hydrodynamical analogy; that is, a flow through pervious materials. The laminar flow through pervious materials is governed by Laplace's equation,

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = O$$

The solution for the flow through pervious soils (pervious material) was presented by Casagrande (3). A. Casagrande based his graphical solution of flow problems on Forchheimer's solution of Laplace-equation (4).

SOLUTION

1. *Laplace-equation and its graphical solution*

Laplace-equation $\frac{\partial^2 a}{\partial x^2} + \frac{\partial^2 a}{\partial y^2} = O$ represents two orthogonal families of curves.

In hydromechanics, these lines are referred to as equipotential or equal-head lines and flow-lines (3, 5, 6). The actual number of lines which can be drawn is infinite, but for the purpose of drawing the "flow-net" (3, 5, 6), only a few are chosen in such a way that the following relationships occur:

1. $\Delta q =$ const. The seepage between two neighboring flow lines is constant.
2. $\Delta h =$ const. The drop of head between two equipotential lines is constant.
3. $\frac{\Delta q}{\Delta h} =$ const.

From the practical point of view, an important characteristic of the family of lines, is such that, if $\frac{q}{h} = 1$, the two families of curves form squares (3).

2. *Presentation of the TORSIONAL PROBLEM in Form of Laplace equation*

To present the torsional problem in the form of Laplace-equation, an algebraic transformation is required.

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = -2G\theta \qquad (4)$$

let $-2G\theta = F$ therefore $\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = F$

and $\phi = \phi_1 + \frac{F}{4}(x^2 + y^2)$ hence $\frac{\partial^2 \phi}{\partial x^2} = \frac{\partial^2 \phi_1}{\partial x^2} + \frac{F}{2}$;

$\frac{\partial^2 \phi}{\partial y^2} = \frac{\partial \phi_1{}^2}{\partial y^2} + \frac{F}{2}$ substituting into equation (4)

$\frac{\partial^2 \phi_1}{\partial x^2} + \frac{F}{2} + \frac{\partial^2 \phi_1}{\partial y^2} + \frac{F}{2} = F$ and finally

$$\frac{\partial^2 \phi_1}{\partial x^2} + \frac{\partial \phi_1{}^2}{\partial y^2} = O \quad \text{which is Laplace equation.} \qquad (5)$$

3. *Boundary Conditions*

The membrane analogy for the equation (1) is

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial h^2}{\partial y^2} = \frac{q}{s}$$

From the analogy we can conclude that ϕ, which is analogous to "h" in membrane, is zero at the boundary and, therefore

$$\phi_1 = -\frac{F}{4}(x^2 + y^2) \quad \text{around the boundary}$$

$\frac{1}{4}\,\frac{q}{s}$ is equivalent to $\frac{F}{4}$ and equals to a constant; therefore, it provides only a scale for the deflected membrane. For simplicity, let us assume $-\frac{F}{4} = 1$. Hence, the magnitude of ϕ_1, at the boundary, equals $(x^2 + y^2)$ or l^2, distance from the origin squared.

4. *Procedure*

a. Select mutually ⊥ axis x and y.

b. Find values of ϕ_1, at the boundary $[\phi_1]_n = l^2{}_n$.

c. Draw a flow-net, as suggested by Casagrande (3).

d. Draw contour lines of a deflected analogous membrane. (ϕ analogous to h).

$$\phi = \phi_1 + 1\,(x^2 + y^2) \qquad (6)$$

e. Find the "scale" of the drawing corresponding to the torque M_T. $M_T = n2\iint\phi\, dxdy = n2V$ where

V—Volume of the deflected membrane

n—Scale factor

therefore, the value of the scale factor will be indicated by

$$n = \frac{M_T}{2V} \qquad (7)$$

f. Find corresponding stresses

$$\tau_{xz} = \frac{\Delta\phi}{\Delta y} \times n \text{ or } \tau_{xz} = \frac{\Delta\phi}{\Delta y}\,\frac{M_T}{2V}\,;\ \tau_{yz} = -\frac{\Delta\phi}{\Delta x}\,\frac{M_T}{2V} \qquad (8)$$

The volume V may be found by multiplying the average area between two contour lines by corresponding $\Delta\phi$. One of the most important features of the graphical presentation is that the maximum stress may be detected easily from the drawn contour lines. In general, the expression for shear in an arbitrary direction is as follows:

$$\frac{\Delta\phi}{\Delta i}\,\frac{M_z}{2V} = \tau_{iz} \qquad (9)$$

The maximum stress will occur at the point of the closest contour lines.

$$\tau_{max} = \frac{\phi_{n+1} - \phi_n}{\Delta l}\,\frac{M_T}{2V} \qquad (10)$$

Where M_T and 2V are constant for a given section and torque.

5. *Examples* (see Figures 1-4)

The examples presented in this paper are chosen so that a comparison may be made with the solutions available in the literature. The sections are simple: square, circular and triangular.

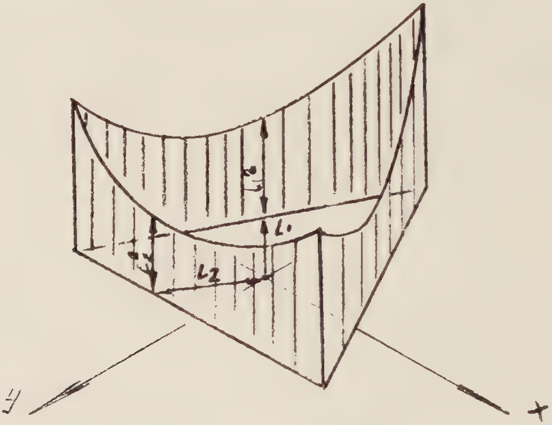


Fig. 1. Model of triangular cross-section.

Sometimes the flow net pattern is quite complicated; therefore, for the purpose of better visualizing of the graphical "picture", the following suggestions are made:

a. Prepare a model of the section in which the boundary condition will be represented by heights of a wall perpendicular to the plane of section. (See Fig. 1).

b. Stretch an elastic membrane at the boundary, by an approximately equal force S.

c. Using a pipette, place a few colored water-solution drops at the boundary. The path of the water flow will draw the corresponding flow lines.

d. Directed by the flow lines on the model, draw the flow net as provided by point c. of the procedure.

Note: No conclusions should be drawn from the errors presented with the examples, for the following reasons:

a. The drawings are sketches only, and

b. The volume V was evaluated approximately.

In general, no attempt was made in the paper to evaluate the magnitude of the error associated with a certain scale. This paper intends only to present the principle of the method, and to suggest a procedure.

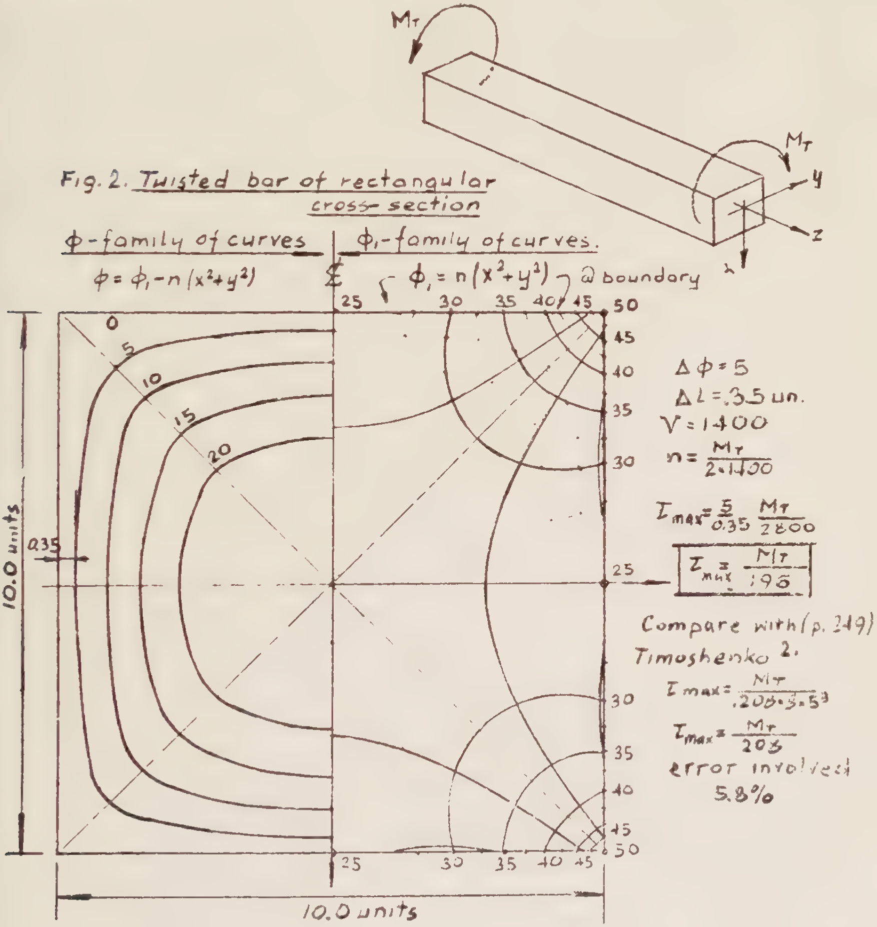


Fig. 2. Twisted bar of rectangular cross-section

6. *Conclusion*

The foregoing method is applicable for every irregular section and may be used wherever a solution is not readily available.

The method further provides a "topographic picture" of stress distribution. It may have special importance for finding regions of stress concentration and the direction of maximum shear stress.

The method may well have special value to a designer for practical purposes, because it may be used with no apparatus. (Which is not the case for soap membrane device.) Obviously, though, the method requires some experience.

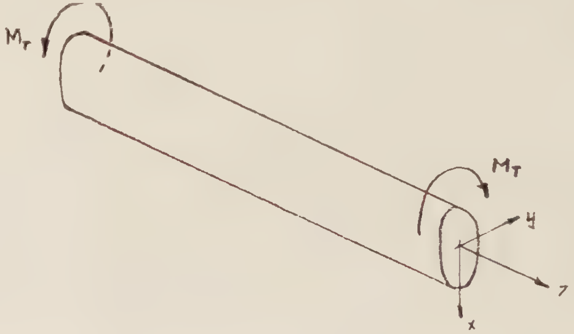
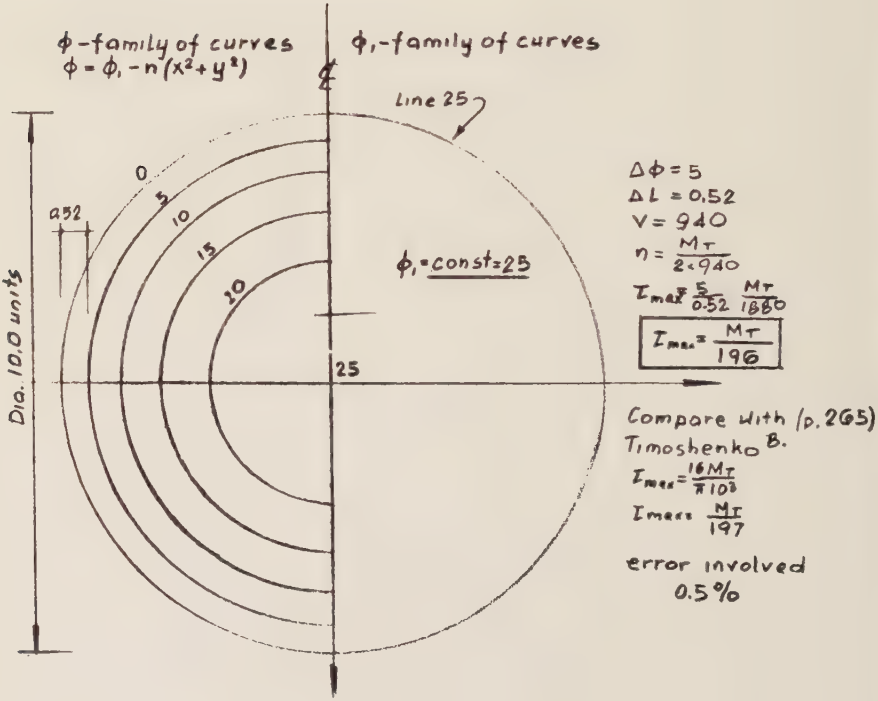


Fig. 3. Twisted bar of circular cross-section



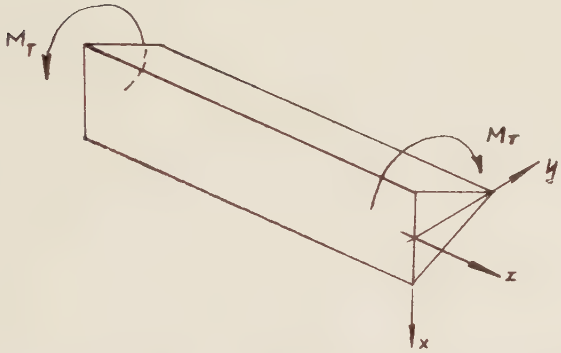
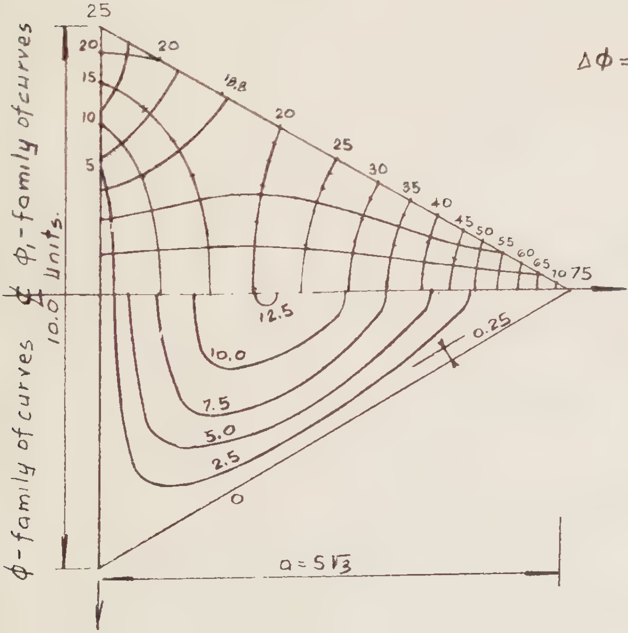


Fig. 4 Twisted bar of triangular cross-section.



$\Delta\phi = 2.5$; $\Delta L = 0.25$

$V = 230$

$$n = \frac{M_T}{2 \cdot 230}$$

$$\tau_{max} = \frac{2.5}{0.25}\,\frac{M_T}{460}$$

$$\boxed{\tau_{max} = \frac{M_T}{46}}$$

Compare with (p. 237) Timoshenko[2].

$$\tau_{max} = M_T\,\frac{15\sqrt{3}}{2 \cdot a^3}$$

$$\tau_{max} = \frac{M_T}{50}$$

error involved 8%

The accuracy of the method depends entirely on the scale of the drawn section. Any accuracy required by the problem, therefore, may be obtained. This is also true for the regions of stress concentration only, as these may be magnified and the flow net subdivided to provide a more exact solution.

In general, the use of the Forchheimer's graphical solution provides unlimited possibilities in the torsional problems.

ACKNOWLEDGMENTS

Generous acknowledgment is made to Professor C. H. Norris of Massachusetts Institute of Technology, who read the manuscript for its evaluation and who provided the author with helpful suggestions for preparation of this paper; to W. L. Thoen, for his criticism and valuable remarks.

NOTATIONS

In general, the notations follow those used by Timoshenko in his books on *Elasticity* and *Strength of Materials*.

τ_{ab} — Shearing stress in "b" direction, acting on a plane perpendicular to "a".
ϕ — Stress function as defined in text.
G — Modulus of elasticity in shear. $\frac{E}{G = 2(1-v)}$
E — Modulus of elasticity.
ν — Poisson's ratio.
θ — Angle of twist per unit length.
M_T — Torque.
n — Scale factor.
V — Volume of deflected membrane.
x, y — Ordinates.

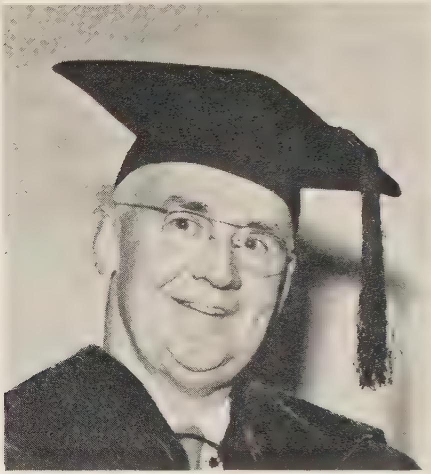
Membrane analogy notations:

h — Lateral deflection of membrane.
q — Lateral uniform membrane loading.
s — Axial uniform membrane loading.

REFERENCES

2. *Theory of Elasticity*, S. Timoshenko, 1st Edition, McGraw-Hill Book Company, New York, 1934.
3. Boston Society of Civil Engineers, *Contribution to Soil Mechanics*, Volume 1, page 295; Boston Society of Civil Engineering, Edition 1940.
4. *Hydraulik*, Philip Forchheimer, 3rd Edition, 1930.
5. *Soil Mechanics in Engineering Practice*, Karl Terzhagi and Ralph B. Peck, John Wiley & Sons, Inc., New York, 1948.
6. *Theoretical Soil Mechanics*, Karl Terzhagi, John Wiley & Sons, Inc., New York, 1948.
7. *Handbook of Experimental Stress Analysis*, M. Hetenyi, John Wiley & Sons, Inc., New York, 1950.
8. *Strength of Materials*, S. Timoshenko, 2nd Edition, D. Van Nostrand Company, Inc., New York, 1951.

OF GENERAL INTEREST



FRANK L. FLOOD

NEWS OF MEMBERS

Frank L. Flood, Partner in the firm of Metcalf & Eddy, Boston, is shown after receiving a hood signifying an honorary Doctor of Engineering degree from Northeastern University at the annual commencement exercises held on June 17, 1955.

PROFESSIONAL SERVICES
AND
ADVERTISEMENTS

The advertising pages of the JOURNAL aim to acquaint readers with Professional and Contracting Services and Sources of Various Supplies and Materials. You would find it of advantage to be represented here.

Volume 42 OCTOBER, 1955 Number 4

BOSTON SOCIETY OF CIVIL ENGINEERS

FOUNDED 1848

PROFESSIONAL SERVICES

PAGE
Listed Alphabetically ii

INDEX TO ADVERTISERS

Algonquin Engraving Co., 18 Kingston St., Boston vi
Beacon Piping Co., 200 Freeport St., Dorchester 22, Mass. x
Berke Moore Co., Inc., 8 Newbury St., Boston ix
Boston Blue Print Co. Inc., 120 Boylston St., Boston xi
Builders-Providence, Inc., Providence 1, R. I. xii
Chapman Valve Mfg. Co., Room 912, 75 Federal St., Boston viii
De Falco Concrete, Inc., Millbury ix
Fletcher, H. E., Co., West Chelmsford, Mass. xiii
Heffernan Press, 150 Fremont St., Worcester xiv
Heinrich Company, Carl, 711 Concord Ave., Cambridge xiii
Johnson, Andrew T., Co., 15 Tremont Place, Boston xi
Makepeace, B. L., Inc., 1266 Boylston St., Boston xiv
National Gunite Corp., 101 W. Dedham St., Boston ix
New England Concrete Pipe Corp., Newton Upper Falls, Mass. vi
New England Power Service Company, 441 Stuart St., Boston viii
Northern Steel Company, 44 School St., Boston viii
O'Connor, Thomas, & Co., 238 Main St., Cambridge vi
Old Colony Crushed Stone Co., Quincy, Mass. x
Pipe Founders Sales Corp., 131 State Street, Boston viii
San-Vel Concrete, Littleton xv
S. Morgan Smith Co., 176 Federal St., Boston xiv
Spaulding-Moss Co., 42 Franklin St., Boston ix
Raymond Concrete Pile Co., Park Square Building, Boston x
Tomasello Corporation, 25 Huntington Ave., Boston vi
United States Pipe and Foundry Company, 250 Stuart St., Boston . . . xi
Warren Brothers Roads Company, Cambridge, Mass. xiv
Warren Foundry & Pipe Corp., 75 Federal St., Boston vii
West End Iron Works, Cambridge xi

Please mention the Journal when writing to Advertisers

Brask Engineering Company

ENGINEERS

177 STATE STREET, BOSTON

CA 7-3170

Clarkeson Engineering Company
Incorporated

Design, Construction Inspection, Management of

Airports, Bridges, Tunnels, Highways, Traffic and Transportation Analyses and Reports Parking and valuations.

285 Columbus Ave., Boston 16, Mass.
Commonwealth 6-7720

CAMP, DRESSER & McKEE

Consulting Engineers

6 BEACON STREET BOSTON 8, MASS.
Telephone CApitol 0422
Water Works and Water Treatment
Sewerage and Sewage Treatment
Municipal and Industrial Wastes
Investigations & Reports Design & Supervision
Research and Development Flood Control

Coffin & Richardson, Inc.

Consulting Engineers

68 DEVONSHIRE STREET

BOSTON 9, MASSACHUSETTS

Congdon, Gurney & Towle, Inc.

Engineers

Structural Design for Architects

Buildings, Bridges, Foundations

53 State Street Boston, Mass.

Cotton, Pierce, Streander, Inc.

Associated Engineering Consultants

Water Supply and Treatment, Sewage Systems and Treatments, Industrial Waste Disposal, Municipal Refuse Disposal, Industrial and Utility Power Plants, Reports, Plans, Design, Supervision.

6 Beacon Street, Boston 8, Mass.
CApitol 7-9465

WILLIAM S. CROCKER
(Formerly Aspinwall & Lincoln)

LOUIS A. CHASE

Registered Professional Engineers
Registered Land Surveyors

46 Cornhill Boston, Massachusetts

IRVING B. CROSBY

Consulting Engineering Geologist

Investigations and Reports
Dams, Reservoirs, Tunnels, Foundations,
Groundwater Supplies and Resources
Non-Metallic Minerals

6 Beacon St., Boston 8, Massachusetts

Richard J. Donovan, Inc.

CIVIL
CONSTRUCTORS & ENGINEERS

Designs
Reports—Surveys
Estimates—Locations
Supervision—Appraisals
Construction—Consultation

Box 186 Winchester, Mass.

Duffill Associates, Inc.

Consulting Engineers

914 BEACON STREET
BOSTON 15, MASSACHUSETTS
CO 6-7070

Edwards, Kelcey and Beck

ENGINEERS

Reports — Design — Supervision
Municipal Works, Highways, Traffic Parking

165 Congress Street, Boston, Mass.
Phone HAncock 6-5330

CRANDALL DRY DOCK ENGINEERS, Inc.

Dry Docks — Piers — Waterfront Structures
Skilled Divers for underwater examination and repair.

238 Main Street Cambridge, Mass.

Fay, Spofford & Thorndike

Engineers

John Ayer
Bion A. Bowman
Carroll A. Farwell
Ralph W. Horne
William L. Hyland
Frank L. Lincoln
Howard J. Williams

Port Developments Foundations Airports
Commercial Buildings Bridges Turnpikes
Industrial Plants

BOSTON NEW YORK

HARRY R. FELDMAN

CIVIL ENGINEER

Engineering and Survey Service for

Consulting Engineers - Architects
Contractors - Appraisers

27 SCHOOL ST., BOSTON, MASS.

Ganteaume & McMullen

Engineers

99 Chauncy Street

BOSTON

GARDNER S. GOULD

Consulting Engineer

Port Developments, Wharves, Piers and Bulkheads, Oil Storage, Coal Handling, Warehouses and Foundations

161 Devonshire St., Boston, Mass.

J. F. HENNESSY

Civil Engineer

4 Cypress Street, Brookline, Mass.

Telephone LO. 6-3860

JACKSON & MORELAND

Engineers and Consultants

Design and Supervision of Construction
Reports — Examinations — Appraisals
Machine Design — Technical Publications

Boston New York

HALEY & WARD

Engineers

Successors to Frank A. Barbour

Water Works and Sewerage
Valuations
Supervision of Construction and Operation

73 TREMONT ST. BOSTON, MASS.

Charles A. Maguire & Associates

Engineers

14 Court Square Boston, Mass.

CHAS. T. MAIN, Inc.

Engineers

80 FEDERAL ST., BOSTON, MASS.

INDUSTRIAL BUILDING DESIGN
STEAM AND HYDRO-ELECTRIC PLANTS
ELECTRICAL ENGINEERING
INVESTIGATIONS—APPRAISALS
FOUNDATIONS

FRANK MARCUCELLA

Engineer

Construction Manager

87 Walsh Street, Medford, Mass.

METCALF & EDDY

Engineers

Water, Sewage, Drainage, Refuse and Industrial Wastes Problems

Airfields Valuations
Laboratory

STATLER BUILDING BOSTON 16

C. J. KRAY

Consulting Engineer

Designs, Reports, Specifications
Valuations, Investigations
Supervision of Construction

HUbbard 2-7240

294 Washington St., Boston 8, Mass.

New England Survey Service Inc.

Civil Engineers - Surveyors

FIRST ORDER SURVEYS

Bridges - General Construction - Highways - Housing - Land Court - Land Takings - Topographical Surveys. Fairchild Aerial Survey Representatives. Aerial Surveys - Oblique Views - Contour Maps.

51 CORNHILL BOSTON 8, MASS.

The Pitometer Associates, Inc.

Engineers

Water Waste Surveys
Trunk Main Surveys
Water Distribution Studies
Water Measurement and Special Hydraulic Investigations

New York 50 Church Street

Herman G. Protze

Materials Technologist

Consultation, Control and Testing on Concrete and Construction Materials

Office and Laboratory

770 HUNTINGTON AVE., BOSTON, MASS.
Tel. Beacon 2-4460

MAURICE A. REIDY

Consulting Engineer

Structural Designs Foundations

101 Tremont Street
BOSTON, MASSACHUSETTS

Gilbert Small & Co.

INCORPORATED

Engineers

Structural Designs

Foundations - Buildings - Bridges

10 STATE STREET BOSTON 9, MASS.

JOHN R. RAND

Consulting Economic Geologist
Mining and Engineering Geology

482½ Congress St., Portland, Maine

THE THOMPSON & LICHTNER CO., INC.

Engineers

Designs and Engineering Supervision
Investigations, Testing and
Inspection of Structural Materials
Concrete Quality Control
Marketing and Production Service

Offices and Laboratory, 8 Alton Place, Brookline 46, Mass.

HOWARD M. TURNER

Consulting Engineer

Investigations, Valuations, Plans, Supervision of Construction, Water Power, Water Supply, Flood Control, Public Utility and Industrial Properties.

6 Beacon Street : : : : Boston

WESTON & SAMPSON

Consulting Engineers

Water Supply, Water Purification, Sewerage
Sewage and Industrial Waste Treatment
Supervision of Operation of Treatment Plants
Laboratory

14 BEACON STREET, BOSTON

Frank H. Whelan & Co.

Consulting Engineers

Real Estate Appraisals
Structural Design

11 BEACON STREET BOSTON, MASS.

WHITMAN & HOWARD

Engineers

(Est. 1869. Inc. 1924)

Investigations, Designs, Estimates, Reports and Supervision, Valuations, etc., in all Water Works, Sewerage, Drainage, Waterfront Improvements and all Municipal or Industrial Development Problems.

89 Broad Street Boston, Mass.

S. J. TOMASELLO CORPORATION

General Contractors
Asphalt Pavements

25 Huntington Avenue Boston 16, Massachusetts

Tel. Kenmore 6-3690

New England Concrete Pipe Corp.

NEWTON UPPER FALLS, MASSACHUSETTS
LAsell 7-4560

MANUFACTURERS OF

Plain and Reinforced Concrete Sewer and Culvert Pipe

PLANTS

Newton, Springfield, Dedham, Massachusetts : : : Providence, Rhode Island

---

THOMAS O'CONNOR & CO., INC.

Structural Engineers and Builders

238 MAIN STREET, CAMBRIDGE, MASSACHUSETTS

Kendall Square

Algonquin Engraving Co., Inc.

PHOTO-ENGRAVERS

18 KINGSTON STREET, BOSTON, MASSACHUSETTS

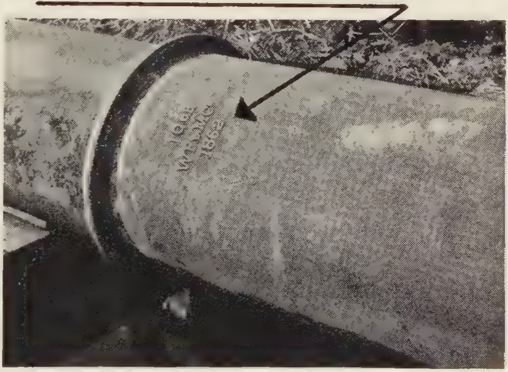
Tel. Han 4855-6

Two Symbols of PERMANENCE *and* DEPENDABILITY

Having withstood centuries of exposure to wind and weather, the Pyramids of Egypt are symbolic of "Permanence".

Similarly, this pipe cast by Warren in 1862 is still in service and has proved its *permanence* and *dependability*.

Over $1,000,000 saved by reusing 93 year old Cast Iron water pipe



While this cast iron pipe was produced in our foundry nearly 100 years ago and serves to give added proof of the *durability* and *permanence* of cast iron pipe installations, this example is by no means either exceptional or even one of the oldest of recorded long-time service.

THE CAST IRON PIPE CENTURY CLUB

This unique club whose membership is made up of those cities in the U. S. A. having cast iron pipe installations of 100 years of age or more, was established by the Cast Iron Pipe Research Association in 1947. Today's membership is 69 and is growing regularly.

BIG SAVINGS

In addition to the value of permanence and uninterrupted service, many cities and communities are realizing sizable dollar savings in instances where extensions or various changes in layouts need be made. With the pipe illustrated, Jersey City, New Jersey, was able to clean, relay and reline the old existing 36-inch pipe at a saving of over $1,000,000 based upon the estimated cost of placing a new line into service. The re-laid main will provide a capacity of 35 m. g. d. of water when required and the City anticipates another century of service from this pipe.

Warren pipe and fittings have been making their contribution to projects and developments for 100 years — literally providing

"A Century of Quality in Service".

WARREN FOUNDRY & PIPE CORPORATION
75 Federal Street, Boston, Massachusetts
EXECUTIVE OFFICES 55 LIBERTY ST., NEW YORK 5, N. Y.
"A CENTURY OF QUALITY IN SERVICE"

PIPE FOUNDERS SALES CORP.

Cast Iron Pipe and Fittings
Dresser Couplings
for
Water, Steam, Gas and Chemical Use
131 STATE STREET : BOSTON 9, MASSACHUSETTS
LA 3-2438

ELECTRICITY IS CHEAP
. . . and there is plenty of it
FOR NEW ENGLAND HOMES AND INDUSTRY

Use all you want — and enjoy it!

NEW ENGLAND ELECTRIC SYSTEM
Means Better Living

NEW ENGLAND POWER SERVICE CO.
PART OF NEW ENGLAND ELECTRIC SYSTEM

The Chapman Valve Manufacturing Co.

Room 912 — 75 Federal Street

BOSTON 10, MASSACHUSETTS

Valves for All Services

Sluice Gates - Shear Gates - Flap Valves

Northern Steel Inc.
1 STATE STREET, BOSTON, MASSACHUSETTS
Concrete Reinforced Bars
Works at
Glenwood Station, Medford, Massachusetts

DE FALCO CONCRETE, INC.

MILLBURY, MASSACHUSETTS

READY MIXED CONCRETE

SAND & GRAVEL

ONE TO A MILLION COPIES – BLACK & WHITE OR COLOR

Satisfied customers are the best evidence we can offer to our success in the photo offset field.

Call us now for your printing needs.

- PHOTO OFFSET LITHOGRAPHY
- BLUE PRINTS • DYELINE PRINTS
- PHOTOSTATS • SENSITIZED PAPERS
- DRAWING — ENGINEERING SUPPLIES AND EQUIPMENT

For Economical Fast Dependable Service

SPAULDING-MOSS CO.

42 FRANKLIN ST. BOSTON 10, MASS.

Phone Liberty 2-3000

BERKE MOORE COMPANY, INC.

General Contractors

8 NEWBURY STREET BOSTON, MASS.

COM 6-0245

NATIONAL GUNITE CORPORATION

Design and Construction of Gunite Tanks, Digesters, Stacks, Walls, Swimpools, Etc. • Concrete Restoration • Encasing Steel • Pressure Grouting • Dam Repairs

101 WEST DEDHAM STREET
BOSTON 18, MASSACHUSETTS

Telephone: COmmonwealth 6-7020

Telephone COlumbia 5-2600

BEACON PIPING COMPANY

Power Plant Piping - High Pressure Piping

Fabricators of Piping

200 FREEPORT STREET
DORCHESTER 22, MASSACHUSETTS

John Fairfield, Pres. H. G. Fairfield, Treas. M. M. Stewart, Gen. Mgr.

Old Colony Crushed Stone Company

Crushed Stone and Bituminous Concrete

Truck and Rail Shipments

OFFICE AND WORKS
VERNON and INTERVALE STS., QUINCY, MASSACHUSETTS
Telephone: Office: PR 3-0604

GOW BORINGS GOW CAISSONS

CONCRETE PILES

RAYMOND CONCRETE PILE CO.

Park Square Building • BOSTON

Telephone HAncock 6-1826

Cast Iron Pipe and Fittings

for

Water, Gas, Sewerage and Industrial Service

UNITED STATES PIPE AND FOUNDRY CO.
250 Stuart Street Boston 16, Mass.

WEST END IRON WORKS
CAMBRIDGE, MASSACHUSETTS

Structural Steel

Fabricators and Erectors

Boston Blue Print Company, Inc.
Boston, Massachusetts
ALL TYPES OF REPRODUCTION PROCESSES
Four Convenient Locations

EXCLUSIVE DISTRIBUTORS
GURLEY ENGINEERING INSTRUMENTS
POST DRAFTING & ENGINEERING MATERIALS

Reproduction processes • *Blueprints*
Drawing Materials • *Photostat Copies*

ANDREW T. JOHNSON CO.

15 Tremont Place, Boston 8, Mass.
Branch 103 Newbury Street

Telephone
CApitol 7-1618, 1619

BLUE CHIP PRODUCTS

PROPORTIONEERS, INC.

feeding and proportioning equipment for solutions . . .

OMEGA MACHINE COMPANY

volumetric and gravimetric feeders for dry and fluid materials . . .

BUILDERS-PROVIDENCE, INC.

equipment for recording, indicating, and controlling. Water pipe fittings.

. . . Call on the New England Sales Office of B-I-F Industries . . .

HENRY M. MAUTNER
Manager

JOHN M. D. SUESMAN
Water Pipe Fittings Sales

GERALD H. LAMPREY
Maine, Northern N. H. & Vt.

RUDOLPH H. GRUNER
R. I., Southern N. H., Eastern Mass.

LORRIN S. BRICE, JR.
Western Mass., Southern Vt.

FRANK A. SHAW
Connecticut

ROBERT F. KELSEY
Boston Office

JULES E. BANVILLE
Providence Office

HOME OFFICE
345 Harris Avenue
Providence 1, R. I.
Telephone: GAspee 1-4302

BOSTON OFFICE
80 Boylston Street
Boston 16, Mass.
Telephone: HAncock 6-1060

B-I-F INDUSTRIES

PROVIDENCE, RHODE ISLAND

METERS
FEEDERS
CONTROLS

RENT with Purchase Option

Transits and Levels

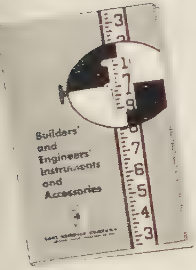
Dietzgen • Sokkisha • David White

Late model, internal focusing, new and like new

Visit our Salesroom or check our Catalog for **PURCHASE** of practically any instrument or accessory you need

. . . most called for products of leading manufacturers:

KERN LUFKIN DIETZGEN
DAVID WHITE KUKER-RANKIN SOKKISHA
LENKER ROLATAPE



Builders' and Engineers' Instruments and Accessories

Complete Illustrated Catalog FREE to Engineers and Builders

Good used instruments bought, sold & traded

CARL HEINRICH COMPANY
711 CONCORD AVE., CAMBRIDGE 38, MASS.
UNIVERSITY 4-4840

FLETCHER granite

for

BRIDGES AND BUILDINGS
STREET AND HIGHWAY CURBING

ASHLAR VENEER

for

BRIDGE FACING - HOUSING
LANDSCAPING

●

GRANITE SURFACE PLATES

Brochure and price schedule will be mailed on request

●

H. E. FLETCHER CO.
QUARRY AND OFFICE WEST CHELMSFORD, MASS.
Phone Glenview 7-7588

HALF A CENTURY OF PROGRESS

It's over a half century—fifty-four years to be exact—since the day the HEFFERNAN PRESS made its feeble start. That each year since then has shown an increase in the volume of business over the previous year should prove that we have kept pace with the times.

THE HEFFERNAN PRESS

150 Fremont Street **Worcester, Massachusetts**

Printers to
Boston Society of Civil Engineers
and OTHER good publications.

S. Morgan Smith Company

Manufacturers of

HYDRAULIC TURBINES

and Accessories

Rotovalves

Axial Flow Pumps

176 Federal Street
BOSTON, MASSACHUSETTS

Only
MAKEPEACE
distributes the famous
K+E IN NEW ENGLAND

Transits, Levels, Tapes & Drafting Instruments.

Come in and choose *all* your engineering equipment from the largest stock in New England.

Complete Repair & Rebuilding Service

BLUE PRINTS • PHOTOSTATS
PLAN REPRODUCTIONS

Call CO 7-2700

B. L. MAKEPEACE INC.
1266 BOYLSTON STREET BOSTON

Asphalt Paving

WARREN BROTHERS ROADS COMPANY

Leaders in design and construction of asphalt pavements since 1902. Engineering staff and testing laboratory available to furnish specifications and advice on unusual paving problems.

Cambridge, Massachusetts **TRowbridge 6-4320**

SAN-VEL

CONCRETE

STANDS READY TO DELIVER

CONCRETE

MIXED TO YOUR SPECIFICATIONS

CALL LITTLETON, MASSACHUSETTS

Hunter 6-3501

COMMITTEES

1955-1956

NOMINATING COMMITTEE

Past Presidents (Members of the Committee)

Harrison P. Eddy, Jr.
Thomas R. Camp
John B. Wilbur

James F. Brittain
Frank L. Lincoln
Henry A. Mohr
(Term expires March, 1956)

Allen J. Burdoin
Fozi M. Cahaly
Eliot F. Childs
(Term expires March, 1957)

SPECIAL COMMITTEES

PROGRAM

Edwin B. Cobb, *Chairman, ex-officio*

George R. Rich
Harold A. Thomas
Edward C. Keane
James Adam, Jr.
Donald W. Taylor
Fozi M. Cahaly
Robert W. Moir
Ariel A. Thomas
Casimir J. Kray
Ralph S. Archibald
Alexander J. Bone

PUBLICATION

Charles E. Knox, *Chairman*

Henry I. Wyner
Dean F. Coburn
Robert L. Meserve
Stephen E. Dore, Jr.
Ariel A. Thomas
Casimir J. Kray
Ralph S. Archibald
Alexander J. Bone

LIBRARY

George M. Reece, *Chairman*

Roland S. Burlingame
Henry R. Wheeler
Clifford S. Mansfield
Lorin M. Pittendreigh
Otto O. Pasanen
Llewellyn T. Schofield
Thomas C. Coleman
Myle J. Holley

HOSPITALITY

Roger C. Collette, *Chairman*

David H. Hamilton
Frank T. Smith, Jr.
Llewellyn L. Cross, Jr.

DESMOND FITZGERALD AWARD

Gordon M. Fair, *Chairman*

John B. Wilbur
John B. Babcock, 3rd.

SANITARY SECTION AWARD

Gordon M. Fair, *Chairman*

Kenneth F. Knowlton
William E. Stanley

STRUCTURAL SECTION AWARD

John B. Wilbur, *Chairman*

Albert Haertlein
Earle F. Littleton

HYDRAULICS SECTION AWARD

Gordon M. Fair, *Chairman*

Ariel A. Thomas
Frank L. Heaney

SURVEYING & MAPPING SECTION AWARD

John B. Babcock, 3rd, *Chairman*

Charles M. Anderson
Paul A. Dunkerley

TRANSPORTATION SECTION AWARD

John B. Wilbur, *Chairman*

John D. M. Luttman-Johnson
George W. Hankinson

SUBSOILS OF BOSTON

Donald W. Taylor, *Chairman*

Miles N. Clair
Irving B. Crosby
Lawrence G. Ropes
Chester J. Ginder

INVESTMENT COMMITTEE

Charles O. Baird, Jr., *Chairman*

Edwin B. Cobb
Donald W. Taylor

MEMBERSHIP CENTRAL COMMITTEE

Joseph C. Lawler, *Chairman*

James L. Dallas
Frank L. Lincoln
James F. Brittain
Chester A. Moore
Robert A. Bierweiler
Edward W. Moore
George W. Hankinson
Percival S. Rice
John M. Biggs
Elliot F. Childs
Robert W. Moir

AUDITING COMMITTEE

Charles H. Norris
Thomas A. Berrigan

ADVERTISING COMMITTEE

John F. Flaherty, *Chairman*

Charles E. Knox
Ferdinand A. Trautner
Herbert E. Fletcher, Jr.
Paul S. Crandall
Benjamin A. Lekesky
Joseph C. Knox
Albert Adelman
John C. Finn

JOINT LEGISLATIVE COMMITTEE
(formerly Committee on Registration Law)

Athole B. Edwards
Edward Wright

PUBLIC RELATIONS COMMITTEE

John H. Hession, *Chairman*

Joseph A. Dunn
Frank H. Dillaby
Frank Marcucella
Thomas C. Coleman
George W. Coffin

INDEX

VOLUME 42, 1955

Names of Authors are printed in Italics

Address at Annual Meeting. Some Thoughts Relative to the Society. *Miles N. Clair* . . . April, 75

Background and Highlights of the Interim Report of the Committee on Evaluation of Engineering Education. *Harold L. Hazen* . . . Jan., 18
Baffle Piers, Effect of on Stilling Basin Performance. *Donald R. F. Harleman* . . . April, 84
Berrigan, Thomas A. Discussion, Ethics in Engineering . . . July, 214
Bowman, Waldo G. Ethics in Engineering . . . July, 207
Bray, Oscar S. Thoughts on an Engineering Education . . . Jan., 35
Bridge Construction, Development of Prestressed Concrete in. *John C. Rundlett* . . . Oct., 293

Camp, Thomas R. Discussion, Ethics in Engineering . . . July, 211
Clair, Miles N. Some Thoughts Relative to the Society. Presidential Address at Annual Meeting . . . April, 75
Cobb, Edwin B. Pollution Abatement Program at Newport, R. I. . . . July, 217
Coburn, Dean F. Pollution Abatement Program at Newport, R. I. . . . July, 217
Crandell, Francis J. Determination of Incipient Roof Failures in Rock Tunnels by Micro-Seismic Detection . . . Jan., 39

Determination of Incipient Roof Failures in Rock Tunnels by Micro-Seismic Detection. *Francis J. Crandall* . . . Jan., 39
Development of Prestressed Concrete in Bridge Construction. *John C. Rundlett* . . . Oct., 293
Development of Structural Flight Testing. *Vice Admiral Frederick M. Trapnell* . . . April, 158
Dietz, Albert G. H. Physical and Engineering Properties of Plastics . . . April, 134
DuBois, Richard. Hydraulic Performance of Check and Control Valves . . . July, 263

Effect of Baffle Piers on Stilling Basin Performance. *Donald R. F. Harleman* . . . April, 84
Ethics in Engineering. *Waldo G. Bowman* . . . July, 207

Graphical Solution for Torsional Problems Hydrodynamical Analogy. *Alexander Wargon* . . . Oct., 364

Hall, Wilfred M. The St. Lawrence International Hydroelectric Development . . . . July, 243
Harleman, Donald R. F. Effect of Baffle Piers on Stilling Basin Performance . . . . April, 84
Hazen, Harold L. Background and Highlights of the Interim Report of the Committee on Evaluation of Engineering Education . . . . Jan., 18
Heaney, Frank L. Improvement to New Bedford's Sewerage Facilities, Including Intercepting Sewers & Pumping Stations . . . . Oct., 323
High-Rate Sludge Digestion. *Clair N. Sawyer and Harold E. Schmidt* . . . . Jan., 1
Humanistic-Social Contact of Engineering Curricula. *William C. White* . . . . Jan., 24
Hydraulic Performance of Check and Control Valves. *Ronald E. Nece and Richard DuBois* . . . . . . July, 263
Hydroelectric Development. The St. Lawrence International. *Wm. F. Uhl, W. M. Hall and G. R. Rich* . . . . July, 243
Hydrodynamical Analogy, Graphical Solution for Torsional Problems. *Alexander Wargon* . . . . . . Oct., 364

Improvement to New Bedford's Sewerage Facilities, Including Intercepting Sewers and Pumping Stations. *Frank L. Heaney* . . . . Oct., 323
Incinerators, Problem of a Metropolitan District System of Refuse Disposal. *C. A. Turner* . . . . Jan., 60

Jones, Robert T. Discussion, Improvements to New Bedford's Sewerage Facilities, Including Sewers and Pumping Station . . . . Oct., 343

Metropolitan District, Problem of Refuse Disposal Incinerators. *C. A. Turner* . . . . Jan., 60

Nece, Ronald E. Hydraulic Performance of Check and Control Valves . . . . July, 263
New Bedford, Improvement to Sewerage Facilities, Including Intercepting Sewers and Pumping Stations. *Frank L. Heaney* . . . . Oct., 323
Newport, R. I. Pollution Abatement Program. *Edwin B. Cobb and Dean F. Coburn* . . . . July, 217

On Some Aspects of the Theory of Thin Elastic Shells. *Eric Reissner* . . . . April, 100

Physical and Engineering Properties of Plastics. *Albert G. H. Dietz* . . . . April, 134
Plastics, Physical and Engineering Properties of. *Albert G. H. Dietz* . . . . April, 134

Pollution Abatement Program at Newport, R. I. *Edwin B. Cobb and Dean F. Coburn* . . . July,
Prestressed Concrete, Development of in Bridge Construction. *John C. Rundlett* . . . Oct.,

R*eissner, Eric.* On Some Aspects of the Theory of Thin Shells . . . April, 100
Rich, George R. The St. Lawrence International Hydroelectric Development . . . July, 243
Roof Failures, Determination of Incipient, in Rock Tunnels by Micro-Seismic Detection. *Francis J. Crandell* . . . Jan., 39
Rundlett, John C. Development of Prestressed Concrete in Bridge Construction . . . Oct., 293

Sanitary Engineering in Alaska. *Clarence I. Sterling* . . . Oct., 345
Sawyer, Clair N. High-Rate Sludge Digestion . . . Jan., 1
Schmidt, Harold E. High-Rate Sludge Digestion . . . Jan., 1
Sewerage Facilities, Improvements to New Bedford's, Including Intercepting Sewers and Pumping Stations. *Frank L. Heaney* . . . Oct., 323
Some Thoughts Relative to the Society. Address at Annual Meeting. *Miles N. Clair* . . . Apr., 75
Sludge Digestion, High-Rate. *Clair N. Sawyer and Harold E. Schmidt* . . . Jan., 1
Sterling, Clarence I. Sanitary Engineering In Alaska . . . Oct., 345
St. Lawrence International Hydroelectric Development. *Wm. F. Uhl, W. M. Hall and G. R. Rich* . . . July, 243
Structural Flight Testing Development. *Vice Admiral Frederick M. Trapnell* . . . April, 158
Synthesis in the Education of the Engineer During Early Training Rather than Later or Not at All. *John B. Wilbur* . . . Jan., 30

Thin Elastic Shells, On Some Aspects of the Theory of. *Eric Reissner* . . . April, 100
Thomas, Harold A. Discussion, Improvements to New Bedford's Sewerage Facilities, Including Intercepting Sewers and Pumping Stations . . . Oct., 341
Thoughts On an Engineering Education. *Oscar S. Bray* . . . Jan., 35
Trapnell, Frederick M. The Development of Structural Flight Testing . . . April, 158
Turner, C. A. Problems of Metropolitan District System of Refuse Disposal Incinerators . . . Jan., 60

U*hl, William F.* The St. Lawrence International Hydroelectric Development . . . Jan., 243

on, Alexander. Graphical Solution for Torsional Problems Hydrodynamical Analogy . . . . Oct., 364
e, William C. The Humanistic-Social Content of Engineering Curricula . . . . July, 243
ilbur, John B. Synthesis in the Education of the Engineer During Early Training Rather Than Later or Not at all . . . . Jan., 30